CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

CONNECTICUT RIVER
CONNECTICUT

ANALYSIS OF DESIGN

1939

CORPS OF ENGINEERS, U.S. ARMY

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

CONNECTICUT RIVER FLOOD CONTROL

ANALYSIS OF DESIGN

EAST HARTFORD DIKE

CONNECTICUT RIVER

CONNECTICUT

CORPS OF ENGINEERS, UNITED STATES ARMY

UNITED STATES ENGINEER OFFICE PROVIDENCE, RHODE ISLAND

EAST HARTFORD DIKE

ANALYSIS OF DESIGN

CONTENTS

EAST HARTFORD DIKE

PERTINENT DATA

Location - Connecticut River, East Hartford, Connecticut

Area protected	3.4 sq.mi.
Limits of dike designed	Sta. 98+00 to 125+50 and Sta. 130+70 to 186+10
Limits of the contract	Sta. 98+00 to 125+50 and Sta. 130+70 to 170+00

Elevations (above mean sea level)

Top of dike (South side railroad bridge)	40.7
Top of dike (Sta. 125+50)	40.5
Top of dike (South side Connecticut Boulevard)	39.9
Top of dike (Sta. 146+30)	39.4
Top of wall at bulk oil terminal (north end)	37.4
Top of wall at bulk oil terminal (south end)	37.4
Top of dike at end of wall	39.3
Top of dike at bend from river (Sta. 170+00)	39.1
Top of dike end at swale (Sta. 186+00)	39.1

Embankment (Sta. 98+00 to 170+00)

Maximum height of dike	30 feet
Total length of dike (Sta. 98+00 to 170+00)	7,200 feet
Total impervious fill	116,000 cu.yds.
Total pervious and random fill	303,000 cu.yds.
Total sheet piling	125,000 sq. ft.
Total riprap (hand placed)	11,500 cu.yds.
Total gravel bedding	10,400 cu.yds.
Total topsoil	11,500 cu.yds.

Concrete Flood Wall

Type	Counterfort
Maximum height (above top of base)	29.4 feet
Total length	567 feet

Concrete quantities

Total	5,050 cu.yds.
Flood wall	4,700 cu.yds.
Miscellaneous structures	350 cu.yds.

I. INTRODUCTION

I. INTRODUCTION

A. AUTHORIZATION AND PAST REPORTS. - The construction under consideration is a portion of the local protection works for the Town of East Hartford, Connecticut, described and included in the project for flood protection of seven localities on the Connecticut River as a part of the comprehensive system for flood control recommended by the District Engineer in "Report of Survey and Comprehensive Plan for Flood Control in the Connecticut River Valley," March 20, 1937, approved by the Chief of Engineers November 29, 1937, and published as House Document No. 455, 75th Congress, 2d Session. The project is authorized under the Flood Control Act approved June 28, 1938.

Both the "Fiscal Year 1939 Section," covered by this Analysis, and the "Initial Fiscal Year 1939 Unit" now under construction were recommended in a subproject dated November 16, 1938, revised December 14, 1938, and approved December 23, 1938, by the Chief of Engineers.

B. GENERAL BRIEF DESCRIPTION OF LOCAL PROTECTION WORKS.

1. Embankment. - The portion of the local protection works as designed will consist of three sections of earth embankment or dike having a maximum height of 30 feet and extending from Sta. 98+00 to Sta. 186+10. It includes a section of concrete flood wall approximately 567 feet long and approximately 400 feet of dike in the "Initial Unit" just upstream of Connecticut Boulevard. See Plate No. 14. Owing to limitations of available funds the work included in the proposed contract is for the portion of the local protection works for East Hartford upstream of Sta. 170+00 only. During floods the earth embankment will be subjected to a maximum head of 25 feet. The dike section is designed with a top

width of 10 feet and side slopes of 1 vertical and 3 horizontal, both sides. A crushed-rock filled drain with subsurface tile drains connected to a drainage system is provided under the landside toe to lower the saturation line through the dike and to collect seepage from the dike, foundation and surface drainage during floods. The main body of the dike will be composed of free draining material supporting an impervious blanket on the riverside slope. The blanket varies in thickness from 3 feet at the top to 5 feet at the bottom of the dike. An exploration and cut-off trench is provided under the riverside toe. Sheet pile cut-off will be constructed under the portions of the dike from Sta. 110+50 to Sta. 116+50 and Sta. 142+50 to Sta. 177+00 where a natural impervious foundation is not available.

2. Concrete wall. - Restricted space in the vicinity of the bulk oil terminal just south of Connecticut Boulevard necessitates the use of a concrete wall. The wall will be of a buttressed type approximately 567 feet long, 20 feet in height above ground, and will have a steel sheet pile cut-off.

II. SELECTION OF SITE

II. SELECTION OF SITE

A. GENERAL LOCATION.

1. Town of East Hartford. - The Town of East Hartford is situated on the east bank of the Connecticut River, 52 miles above the mouth and directly across the river from the City of Hartford. It covers an area of 18.2 square miles and had a population of 17,125 in the 1930 census. The principal business activities are the manufacture of aeroplanes and aeroplane engines, the distribution and sale of petroleum products, fertilizers, foodstuffs, automobiles, and building supplies. For location see Plate No. 1.

2. Description of the flooded area. - The area subject to most frequent flooding is an alluvial plain about 3/4 of a mile wide having an elevation near the Connecticut River of about 20 feet above mean sea level and sloping gently downward to a swale at the foot of a bluff. The river is influenced by tides at this point. The water surface varies from Elevation 2.0 m.s.l. at low flows to a maximum recorded height of 37.5, which occurred during the 1936 flood. See Plates Nos. 2 and 3. A portion of the thickly settled area above the bluff was flooded for the first time during the flood of 1936. The development above the bluff consists of a few small industries, commercial establishments, and residences and apartment houses of well-built, medium-cost construction. The development on the low plain is principally low cost residences, marine ways, river sand and gravel companies, boat clubs and a bulk oil terminal.

The principal thoroughfare, Connecticut Boulevard, crosses the middle of the low plain on an earth fill. The Boulevard is lined with stores, automobile salesrooms, construction equipment storage plants and dwellings, all of which were flooded during the 1936 flood.

3. Existing dikes. - There are no existing flood protection works at East Hartford.

B. SELECTION OF ALINEMENT.

1. Ultimate dike alinement. - The ultimate dike alinement when completed will commence at a point near the intersection of Green Terrace and Main Street, thence westerly 3,500 feet to the bank of the Connecticut River. From this point the dike follows the bank of the river southerly 3,000 feet until it reaches the railroad embankment. The dike begins again on the south side of the railroad embankment and continues 3,100 feet along the bank of the river to high ground at Connecticut Boulevard. The dike begins again just south of Connecticut Boulevard and runs 4,000 feet along the bank of the river to a point a short distance south of Hartland Street where it turns from the Connecticut River and goes easterly 6,400 feet, crossing Main Street, and then follows the Hockanum River until it reaches high ground near the intersection of Brewer Lane and Saunders Street. Stop-log structures will be provided at the New York, New Haven and Hartford Railroad and at the Main Street crossing near the Hockanum River. No stop-log or sand-bag opening is required at Connecticut Boulevard as the roadway is above grade.

2. Width of foreshore vs. damages. - The alinement of the dike was selected after careful study of the existing topography. The factors considered in addition to general economics were (a) safety of the dike; (b) costs of maintenance; and (c) the relation between construction costs to the Government and damage costs to the Town of East Hartford.

In considering the safety of the structures, a wide foreshore was desirable to allow for erosion of the banks, to create a longer seepage path and to provide increased stability against foundation failure. A generous width of the foreshore could not be obtained throughout owing to the fact that the locality is well built up over most of the dike alinement along the Connecticut River. Erosion of the river banks is not a serious consideration for this portion of the dike work as the main channel hugs the farther or west bank of the river. A number of sand dredging industries have been established on the East Hartford side of the river between the two bridges for many years. Two companies are still operating. They have been removing sand from the river over a period of many years and although thousands of yards of sand have been taken away, no great holes have resulted as the material is obviously replaced by the spring freshets. This would indicate that the bank and bed of the river in this stretch have a tendency toward deposition rather than erosion. See Paragraph IX E 1 (Riprap).

3. Limits of the work. - The portion of the dike selected for immediate construction extends from Sta. 98+00 ± at the embankment of the New York, New Haven and Hartford Railroad to Sta. 170+00 where the dike turns easterly from the bank of the Connecticut River, with the exception of a section 400 feet long from Sta. 125+50 to Sta. 129+50 now under construction by hired labor.

III. GEOLOGICAL INVESTIGATIONS

III. GEOLOGICAL INVESTIGATIONS

A. NATURE OF VALLEY. - The Connecticut River at East Hartford is entrenched in a flood plain which extends back from the river a distance of about three-fourths of a mile. Towards the north, above the New York, New Haven and Hartford Railroad bridge, the river is actively eroding its left bank so that the width of the plain has been reduced to about a third of a mile. This alluvial valley bottom, throughout much of its extent, stands at an elevation between 15 and 18 feet above mean sea level. In the extreme easterly portion there is an elongated depression or swale at about Elevation 12. In time of flood this serves as a natural floodway, carrying flood waters into the tributary Hockanum River, near its confluence with the Connecticut River. The higher ground between the swale and the river bank is highly developed. This condition affects not only the location of the dike but also eliminates any possibility of obtaining embankment materials near the site.

B. METHOD AND EXTENT OF EXPLORATIONS. - Subsurface explorations were accomplished by means of core borings, auger borings and test pits. Investigations by core borings, utilizing drive sampling methods, were made (1) to determine the character and thickness of overburden forming the dike foundations, (2) to investigate sediments in the Connecticut River as a possible source of pervious embankment material, (3) to develop further the thickness of overburden in Borrow Area E. Auger borings were utilized wherever practicable in the investigation of borrow areas and the character and thickness of natural soil blanket, overlying pervious strata, in back of the dike. Test pits were used chiefly in Borrow Area E, and in explorations of the foundation, particularly along stretches where the dike will rest on artificial fills. The location of points of exploration are shown on Plate No. 4, entitled "Subsurface Explorations." The records are shown on Plates Nos. 5, 6, and 7 entitled "Record of Subsurface Explorations."

C. SITE. - Artificial fill material occurs in the upper portion of the foundation throughout much (about 60%) of the area between the Memorial Bridge and the New York, New Haven and Hartford Railroad Bridge. This fill is extremely heterogeneous, being made up of sand, gravel, and silt with varying amounts of brick, cinders, and other debris. The upper portions of the foundation, where there is no fill, is composed of fine sand and silt strata, chiefly Classes 6 and 8, varying in thickness from 2 feet to about 25 feet. Underlying this naturally deposited stratum is a pervious formation composed chiefly of coarse and medium sand, Classes 2 and 4. The geologic section indicates that the area of this formation exposed to seepage from the river south of the Memorial Bridge or Connecticut Boulevard is twice that exposed north of this point. The thickness varies from about 10 feet at Sta. 95+00 to about 40 feet at Sta. 180+00. Below the pervious strata are fine silt and clay materials, chiefly Class 12, which are developed throughout the foundation to a depth of 40 feet or more. The distribution of these formations is shown on Plate No. 8 entitled "Geologic Section."

D. NATURE OF EXCAVATIONS. - Excavations, exclusive of those for embankment materials, will be made for toe trenches and stripping of topsoil throughout the foundation area. In those stretches where the upper portions of the overburden are comprised of naturally deposited sand and silt, riverside toe trench excavations will be carried to a depth of about

5 feet. Where the overburden is composed largely of foreign material (fill) the excavations will be carried to a maximum depth of about 20 feet, depending upon the location of naturally deposited materials. Landside toe excavations will be made chiefly in sand and silt, the depth varying between 2 feet and 9 feet.

E. SUBSURFACE LEAKAGE. - As indicated on Plate No. 8 entitled "Geologic Section," the dike foundations can be subdivided into three distinct zones of varying permeability. These are shown as impervious (Class 12), pervious (Classes 2 and 4), and moderately impervious (Classes 6 and 8) formations. Direct contact of moderately impervious formations with the river will be prevented by construction of the impervious toe and riverside blanket. These deposits extend easterly for a considerable distance and will serve as a landside blanket to retard upward seepage and prevent excessive piping. Seepage through the pervious formation between Sta. 110+50 and Sta. 116+50 and south of Sta. 142+00 will be prevented by a sheet pile cut-off. From a leakage standpoint this construction is warranted because of the increased contact area of pervious formations with the river and the inadequacy of the natural landside blanket.

IV. FLOOD HYDRAULICS

IV. FLOOD HYDRAULICS

A. DESIGN FLOOD. - The design flood on which the dike grade is based is the maximum predicted flood reduced by the 20 reservoirs included in the Comprehensive Plan. The determination of the maximum predicted flood is discussed in Appendix 1 of "The Report of Survey and Comprehensive Plan for the Connecticut River," dated March 20, 1937. It has a peak discharge at Hartford of 318,000 c.f.s., approximately 10 per cent greater than the maximum flood of record. See Paragraph VIII A.

B. FREEBOARD. - The survey report proposed a uniform freeboard of 3 feet for both concrete walls and earth embankment. This was based on considerations of wave fetch and velocity. The Board of Engineers for Rivers and Harbors recommended that, since the entire reservoir plan might not be effective for some time, the earth section be raised 2 feet. The freeboard as designed is therefore 5 feet for the earth dike and 3 feet for the concrete walls.

C. LOCAL CONDITIONS. - During major floods a portion of the discharge leaves the main river channel, flows down the East Hartford swale, and rejoins the river below East Hartford, thus dividing the Town of East Hartford into two parts. It is estimated that in a major flood such as that of 1936 as much as 20 per cent of the total flow at the time of peak discharge is by-passed through the swale. Upon completion of the entire proposed East Hartford dike, the flow down the swale will be cut off, and all the flow will be confined in the main river channel, between the Hartford dikes on the right side of the river, and the East Hartford dikes on the left. Flood stages between the two cities and at points upstream will be somewhat higher because of this confinement. These increases in flood heights were considered in fixing the dike grades. The dike to be constructed under the proposed contract will not interfere with the flow of flood waters through the swale.

The Hartford dikes on the right side of the river are being constructed with a top elevation approximately 6 feet above the official grades established by the Board of Engineers for Rivers and Harbors. The additional costs of raising the elevation of the local protection works is being contributed by the City of Hartford.

V. LABORATORY AND FIELD INVESTIGATION OF SOILS

V. LABORATORY AND FIELD INVESTIGATION OF SOILS

A. CLASSIFICATION OF MATERIALS. - The Providence District early adopted a system of soil classification having rigidly standardized terms. In this classification soils are divided into 16 classes, the distinction between each class being based upon size limitations. These classes are shown graphically on Plate No. 12 and described in the table on the following page.

Even numbers are used to designate materials of uniform texture. Odd numbers designate materials of variable texture. The sizes for gravel, sand, silt and clay are the same as those in the so-called M. I. T. classification with the single exception that the size demarcation between fine silt and coarse clay is not rigidly held at 0.002 mm. but is allowed to vary from 0.006 mm. to 0.0006 mm.

B. GRAIN-SIZE ANALYSIS. - Grain-size curves of the samples were obtained by means of sieve and hydrometer analyses. The materials represented by these curves were carefully classified and similar classes grouped into large sedimentary units as shown on Plate No. 8 titled "Geologic Section." Full use was made of every sample recovered in order to show as accurately as possible the actual geological occurrence.

C. WATER CONTENT AND VOID RATIO. - Numerous undistrubed samples were recovered from deposits in the proposed dike foundations. The careful manner by which these samples were obtained made it possible to accurately determine the water content and void ratio of the material in its natural state. Void ratio and water content determinations were also made on samples recovered from borrow areas.

PROVIDENCE SOIL CLASSIFICATION
U.S. ENGINEER OFFICE
PROVIDENCE, R.I.

TABLE NO. 1

CLASS	DESCRIPTION OF MATERIAL
1	Clean Gravel. - Contains little coarse to medium sand.
2	Uniform Coarse to Medium Sand. - Contains little gravel and fine sand.
3	Variable - Graded from Gravel to Medium Sand. - Contains little fine sand.
4	Uniform Medium to Fine Sand. - Contains little coarse sand and coarse silt.
5	Variable - Graded from Gravel to Fine Sand. - Contains little coarse silt.
6	Uniform Fine Sand to Coarse Silt. - Contains little medium sand and medium silt.
7	Variable - Graded from Gravel to Coarse Silt. - Contains little medium silt.
8	Uniform Coarse to Medium Silt. - Contains little fine sand and fine silt.
9	Variable - Graded from Gravel to Medium Silt. - Contains little fine silt.
10	Uniform Medium to Fine Silt. - Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10 C	Uniform Medium Silt to Coarse Clay. - Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11	Variable - Graded from Gravel or Coarse Sand to Fine Silt. Contains little coarse clay.
12	Uniform Fine Silt to Medium Clay. - Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12 C	Uniform Clay. - Contains little silt. Possesses behavior characteristics of clay.
13	Variable - Graded from Coarse Sand to Clay. - Contains little fine clay (colloids). Possesses behavior characteristics of silt
13 C	Variable Clay. - Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

D. PERMEABILITY. - Table No. 2, shown below, indicates the range limits of permeability for each class of overburden material.. These values were determined from laboratory tests of both undisturbed and disturbed samples. The table has its principal use in serving as a guide for evaluating relative permeabilities of these materials.

TABLE NO. 2

General Type	Class	Coefficient of Permeability k x 10-4 cm./sec.	Coefficient of Permeability k x 10-4 ft./min.
Uniform	2	120 to 400	240 to 800
	4	20 to 120	40 to 240
	6	5 to 20	10 to 40
	8	1 to 5	2 to 10
	10 or 10C	0.1 to 1	0.2 to 2
	12 or 12C	Less than 0.1	Less than 0.2
Variable	1	greater than 1000	greater than 2000
	3	200 to 1000	400 to 2000
	5	50 to 200	100 to 400
	7	15 to 50	30 to 100
	9	3 to 15	6 to 30
	11	0.2 to 3	0.4 to 6
	13 or 13C	Less than 0.2	Less than 0.4

E. SHEAR AND COHESION. - Investigations in the field and laboratory have been directed toward obtaining information on the shearing resistance of foundation and borrow pit materials.

Numerous shear tests using the Constant Strain Method have been performed on materials intended for the pervious and impervious sections of the embankment. Values of ∅ ranged from 32° 00' to 36° 00'. These tests were performed on materials screened through the 10 mesh sieve, the results of which are shown on Plate No. 13.

F. COMPACTION. - Many compaction tests based on the Proctor analysis procedure were performed in the laboratory on materials intended for pervious and impervious embankment construction. The result of these tests for each type of embankment material are tabulated below:

TABLE NO. 3

COMPACTION TESTS

	PERVIOUS	IMPERVIOUS
Dry Comp. Wt. #/cu.ft.	111.0	110.4
Optimum Moisture Content	14.4%	17.9%
Minimum Void Ratio	0.552	0.571

G. COMPRESSIBILITY. - The nature of the earth materials in the foundation of the dike indicates that there will be but little settlement and no lateral displacement.

H. OTHER TESTS. - Numerous other tests of less importance included Atterberg limits, extraction for solubility, and specific gravity.

J. BORROW AREAS. - Three borrow areas are proposed as shown on Plate No. 10, entitled "Borrow Areas." The principal source of impervious materials is Borrow Area "E" located in a thickly wooded section approximately 4-1/2 miles east of the center of operations. The formation is composed of compact, mixed materials graded from gravel and coarse sand to fine silt, chiefly Classes 9, 11 and 13.

The natural water content is only slightly above that necessary for maximum compaction. No trouble should be experienced in placing this material. The quantity available is in excess of that required for construction.

Under favorable conditions, Borrow Area "C" may be used as a source of impervious materials. It is located approximately 1-1/2 miles north of the center of operations near the mouth of the Podunk River. The quantity available is in excess of that required for construction.

This area is only slightly above river level, and at times of high water is flooded. Due to the impervious nature of the material (largely Classes 8 and 10), the water content remains high for some time after the flood waters recede, and trouble would be experienced in obtaining proper compaction. For this reason, use of these materials may be prohibited at certain times.

Borrow Areas "A_1", "A_2", and "A_3", located in the bed of the Connecticut River adjacent to the dike are the available sources of materials for pervious embankment construction. These river sediments are composed of coarse to fine sand, Classes 2 and 4, interstratified with beds of mixed materials graded from gravel to fine sand and coarse silt, chiefly Class 5. They occur to a depth of from 5 feet to 15 feet below river bottom. The quantity available is far in excess of that required for construction. Cross sections of these areas are shown on Plate No. 9, entitled "River Cross Sections."

Amounts of materials available and their suitability are summarized in Table No. 4. Grain-size curves of typical materials in Areas A, C and E are shown on Plate No. 11.

TABLE NO. 4

BORROW MATERIALS AVAILABLE

Borrow Area	Occurrence	Volume Available cu. yds.	Volume Required cu. yds.	Intended Use	Test Data Indicating Suitability: Classification	Natural Water Content	Optimum Water Content	Permeability Coef. $k \times 10^{-4}$ cm./sec.	Typical Grain-size Distribution
A1 A2 A3	Varies between 0-5 ft. and 0-15 ft.	1,000,000+	303,000	Pervious section of dike	Classes 2 and 4, some 5	Under Water	None	150 - 400	See Plate No. ___
C	0-4 ft.	140,000±	116,000	Impervious section of dike	Classes 8 and 10	38%±	20% - 22%	0.1 - 1.0	See Plate No. ___
E	At least 0-6 ft.	170,000+	116,000*	Impervious section of dike	Classes 9, 11 and 13	14%±	12%±	0.01 - 0.1	See Plate No. ___

*Alternate to Area C

VI. DESIGN CRITERIA

VI. DESIGN CRITERIA

General design criteria for the East Hartford Dike include safety, stability and reduction of seepage. Statements of various stability and safety requirement conditions to which the dike will be subjected and its ability to resist these conditions are made in Sections IV, VII, VIII, and IX, thus demonstrating that the dike meets the following specific design criteria:

(1) The crest of the dike is at such a grade that there is no danger of overtopping when the design flood occurs. (See Paragraph IV, A).

(2) The freeboard is sufficient to greatly reduce the danger of overtopping by waves. (See Paragraph IV B).

(3) The slopes of the dike are such that, with the materials used in construction, they will be stable under all conditions. (See Paragraphs VII A and IX E).

(4) The line of saturation is well within the land-side toe. (See Paragraphs VII B and IX F).

(5) Water which passes through and under the dike will, when it comes to the surface, have a velocity so small that it is incapable of moving any of the material of which the dike or its foundation is composed. (See Paragraphs VII B, IX F and IX H 1).

(6) There is no possibility for the free passage of water from the riverside to the land-side face. (See Paragraphs VIII B 3 and IX J).

(7) No material soluble in water is used in any part of the dike. (See Paragraph V H).

(8) The foundation is sufficiently stable to resist undue stresses caused by the embankment load. (See Paragraphs II B 1, IX H).

(9) Seepage through the dike and its foundation will be reduced to a total quantity well within economic pumping limitations.

VII. GENERAL DESIGN

VII. GENERAL DESIGN

A. STABILITY OF DIKE. - The design of the embankment in general is based on standard sections adopted by the Providence District and found to be particularly suitable for this Connecticut River Area. The standard sections were adopted after considerable stability analysis determined that they were stable under the worst possible conditions of sudden drawdown of river, lubrication of soil particles in the riverside slope and inappreciable cohesion between the particles. (See Section IX, Paragraph E). Typical cross sections of the dike as designed are shown on Plates Nos. 15, 16 and 17. (See Section VI).

B. PERCOLATION THROUGH DIKE AND FOUNDATIONS. - Some seepage will occur through the dike, but owing to the marked difference in permeability between the impervious blanket on the riverside and the free draining pervious body of the dike, the line of saturation will be well within the land side toe, and any seepage flow will pass harmlessly into the toe-trench drain. The above deductions serve to satisfy design criteria numbers 4 and 5, as mentioned in Section VI. The dike will be founded entirely on soil. (See Geologic Section, Plate No. 8). Throughout most of the length of the dike, there exists at the surface a layer of impervious materials, Classes 6 and 8. At certain points where the layer is thin, or absent, the cut off is taken care of by steel sheet piling. The quantity of seepage is expected to be quite small. The rock-fill toe drain and drainage system are adequate to carry away the seepage.

VIII. HYDRAULIC DESIGN

VIII. HYDRAULIC DESIGN

A. DESIGN GRADE AND FREEBOARD. - The elevations to which the dike and wall have been designed were based on the greatest predicted flood, as modified by the Comprehensive Plan of twenty reservoirs, plus a design freeboard of approximately 5 feet for embankments and 3 feet for concrete walls, as recommended by the Board of Rivers and Harbors.

The following table lists the adopted grades for the tops of the dikes. Grades are straight lines between the stations shown. These grades apply to all of the dikes included in the ultimate dike protection for East Hartford.

(See Table on following page)

TABLE NO. 5

DESIGN GRADES

Location	Dike Type	Report Station	New Station	Earth Grade	Concrete Grade
High ground near Green Terrace	Earth	-	0+00	41.4	-
" " N. of R.R. Bridge	"	-	98+00	41.1	-
" " S. " " "	"	30+00	98+00	40.7	-
(intermediate)	"	45+00	113+00	40.6	-
High ground N. of Conn. Blvd.	"	57+00	129+50	40.5	-
" " S. " " "	"	61+50	130+70	39.9	-
(intermediate)	"	70+00	139+20	39.5	-
(intermediate)	"	75+20	144+70	39.4	-
At N. of bulk oil terminal	Concrete	75+20	144+70	-	37.4
" S. " " " "	"	79+40	148+90	-	37.3
(intermediate)	Earth	79+40	148+90	39.3	-
Bend in dike at Conn. River	"	95+00	170+00	39.1	-
At swale, West of Main Street	"	-	-	39.1	-
Wall, West of Main St. Bridge	Concrete	-	-	-	37.1
" East " " " "	"	-	-	-	37.1
Along Hockanum River, East of Main Street	Earth	-	-	39.1	-
High ground near intersection of Brewer Lane and Saunders Street	"	-	-	39.1	-

B. DRAINAGE SYSTEMS

1. Sewers

a. General. - Provision has been made to collect the seepage passing through and under the dike together with local runoff and discharge it back into the river by gravity ordinarily and by means of pumping stations during flood periods. The seepage is concentrated in the toe drains on the landside of the dike. The toe drains are intercepted at intervals by short lengths of perforated tile pipe which are connected by laterals to a common collector drain located outside the toe of the dike. The collector drains empty into two outfalls, one at Cherry Street, the other at Pitkin Street which in turn discharge into the Connecticut River. The plans provide for future pumping stations at these outfalls.

b. Amount of flow. - Computations were made to determine the probable amount of seepage under and through the dike and the local runoff that could be expected. These quantities were increased materially in order to provide adequate factors of safety and for other local runoff connections in the future. Where the dike intercepts concentrated natural surface drainage, catch basins are provided. It is anticipated that water from other low places adjacent to the dike will find its way into the drainage system through the toe drains. Existing storm and sanitary sewers are intercepted by the collector drains. The sewers have sufficient capacities to take care of storm runoff, sanitary flow and seepage.

c. Collector drains. - The collector drains, of tile and concrete pipe, run parallel to the landside toe of the dike between the dike and the right of way line. Comparative estimates were made to investigate the practicability of placing the trench for the collector drain under the toe trench and using rock for backfill. The greater costs and doubtful benefits caused this method to be abandoned. The Town of East Hartford will contribute the costs of intercepting their storm sewers and the proportionate part of the drainage system due to such interception.

2. Future pumping stations.

a. Purpose. - The drainage area enclosed by the completed dikes is divided into three constituent drainage areas, each requiring a pumping station. These pumping stations are to serve a two-fold purpose. With the completion of the ultimate plan for flood protection of East Hartford, special means will be necessary to dispose of all water and sewage collected within the diked area including seepage. At normal river level the discharge can be made by gravity flow but in times of flood it will be necessary to pump against the high water. It is not proposed to construct the pumping stations in this portion of the project. It was necessary to choose a site and establish the capacity of the Cherry and Pitkin Street stations to design the by-passes which will take care of the discharges by gravity flow until such time as the pump stations will be built. The Town has now under consideration a scheme whereby a central works located within the proposed ultimate diked area at the south end of the swale would provide treatment for all sanitary sewage from East Hartford. Town officials are of the opinion that such a plan will be adopted in the near future.

b. Selection of sites. - In laying out the general scheme for the flood protection of East Hartford three pumping stations were deemed necessary considering the topographic features in the areas involved. In the study for locating the pump covering the area between the railroad and the Connecticut Boulevard the reasons for choosing the site were economy in the drainage system and lack of space for a better site for the pump. In the study of the area south of the Connecticut Boulevard and west of the swale, various sites for the second pump were considered. Influencing the choice of the Pitkin Street site were practicability of grades in the drainage systems flowing to this site and mainly the added factor of safety secured by placing the outfall through the concrete flood wall and the sheet pile cut-off. The site for a third pumping station has tentatively been selected at the south end of the "swale" and is a future consideration. At present the design of the three pumping stations has not progressed beyond the preliminary stage.

c. Capacity.

(1) Cherry Street pumping station. - This station has tributary to it an area of 35 acres. As in the case of the Pitkin Street station, the pumps will be required to handle the storm runoff, the sanitary sewage from the existing sewer system, and the seepage through the dike. A study of the present sewer system has been made, and the effect of future development upon the drainage problem in this area has been investigated.

The present storm water drains as installed by the Town of East Hartford are adequate under existing conditions, and according to local authorities will not have to be enlarged to meet future development. Sanitary sewers have ample capacity to serve areas tributary thereto for present and future conditions. The maximum carrying capacity of both storm and sanitary sewers is 7.4 c.f.s. Drains being installed for the purpose of carrying seepage from the dike and surface drainage from miscellaneous isolated areas have a maximum carrying capacity of 21.6 c.f.s. Installed pumps will have a minimum capacity of 30 c.f.s. divided between two units capable of delivering this discharge under a head of 26 feet, the maximum head against which it will be necessary to pump.

This capacity equals a run-off of 0.85 inches per hour, which is not considered excessive for such a small area, as there will also be dike seepage to be pumped.

(2) Pitkin Street pumping station. - The pumps installed at this station will be required to handle the storm drainage from 57 acres, the sanitary sewage from the existing sewer system, and the seepage through the dike. In the determination of the inflow to be expected at this station, a study of existing drainage systems was made. Consideration was given to future development of the area with its effect upon the run-off characteristics. Additional areas, not now served by sewer systems, but isolated upon completion of dike, were included as tributary to the pumping station.

The present storm water drains as maintained by the Town of East Hartford are adequate under existing conditions, and according to local authorities will not have to be enlarged to meet future development. Sanitary sewers have ample capacity to serve areas tributary thereto for present and future conditions. The maximum carrying capacity of both storm and sanitary sewers is 29.6 c.f.s. Drains being installed for the purpose of carrying seepage from the dike and surface drainage from miscellaneous isolated areas have a maximum carrying capacity of 13 c.f.s.

Installed pumps will have a minimum capacity of 45 c.f.s., divided between two units capable of delivering this discharge under a head of 26 feet, the maximum head against which it will be necessary to pump.

This capacity equals a run-off rate of 0.8 inches per hour, which is not considered excessive for such a small area as dike seepage must also be pumped.

(3) Main pumping station. - Studies to determine the pump capacities at this site are incomplete. The site has not been chosen, except that it will be at the south end of the swale, and additional field work will be necessary to permit an investigation of anticipated flow of sewage and storm water and available storage capacity.

3. Outfalls.

a. General. - It was considered desirable to construct as few conduits through or under the dike as practicable so as to reduce to a minimum the danger of failure through piping. After consideration of every alternative offering reasonable merit, two outfalls were designed consisting of cast iron pipes passing under the dike. Existing outfalls and various small sewer lines will be discontinued as mentioned in Paragraph IX J.

b. Cherry Street. - Upon completion of a study of drainage problems between the railroad and Connecticut Boulevard, it was found that the most practicable location for an outfall was at the Cherry Street site. Provisions were made to intercept the existing storm and sanitary systems belonging to the Town, and divert them into the proposed system near the intersection of Cherry and Village Streets. The size of pipes and grades established are such that the maximum possible flow, including rainfall and sewage can be discharged by gravity to the pump site. During low stages in the river, the sewers have gravity flow into the river. At flood stage pumping will be necessary. Provisions have been made in the system for the installation of a pumping station at this site when the dike system is complete. A flap valve at the outlet and a gate valve at the landside toe of the dike have been provided to prevent backwater flow at flood stages. A low head sluice gate is provided at the future pumping station site to prevent gravity flow through the future pumps. A bronze trimmed gate valve will be installed in such a location as to direct the pump discharge into the outfall conduits and thence into the river. All valves are enclosed in suitable concrete chambers. The outfall conduit passing under the dike has been given the same degree of consideration usually accorded a reservoir outlet through a dam. The pipe, of cast iron, Class C, American Water Works Association, is considered adequate to withstand the embankment loads without collapse. The trench is excavated in original foundation material. A concrete cradle extending throughout the length of the cast iron pipe is provided to evenly distribute the foundation reaction and also to fill the cavity under the pipe which is difficult to eliminate by ordinary backfill methods. Seep rings are provided as an added factor of safety against piping. The conduit passes through the sheet pile cut-off and is joined to it by a concrete ring. See Plate No. 22. Attention is invited to a report of the partial failure of the Table Rock Cove Dam, and also to recommendations regarding conduits passing through earth dams, by Joel D. Justing in "Earth Dam Projects," pages 14 and 166.

c. Pitkin Street. - The outfall conduit, valves and provision for future pumps, etc., at Pitkin Street are similar to those at Cherry Street, the main difference being that the gate valve for emergency use is in a chamber constructed integrally with the concrete flood wall instead of at the landside toe of the dike.

IX. EMBANKMENT AND FOUNDATION TREATMENT

IX. EMBANKMENT AND FOUNDATION TREATMENT

A. MATERIALS AVAILABLE. - The materials available for embankment construction and their suitability have been discussed in detail in Section V, Paragraph J.

B. ECONOMY OF CONSTRUCTION. - Borrow area investigation and construction costs based on dike work done by this District in the vicinity of East Hartford indicates that pervious borrow material can be obtained most economically by dredging from the river. Factors contributing to the selection of dredged borrow material are noted below.

1. The source of borrow material for the pervious embankment is located conveniently in the bed of the Connecticut River adjacent to the dike.

2. Ample water is available for sluicing operations.

3. Water content of the material does not affect the continuity of operations as would be the case with rolled material from dry borrow pits.

4. All materials sluiced are used to the best advantage.

5. There is insufficient volume of borrow material for rolled pervious embankment within reasonable access.

6. The materials from the river bed will have a more favorable coefficient of permeability, thus rendering the dike safer.

The pervious materials dredged from the river bed (Borrow Area A) may be disposed of on shore in one of two ways. One method includes disposal of dredged materials in stock-piles from which pervious materials will be obtained and placed in the embankment by rolled fill methods.

The alternative method includes pumping of dredged materials directly to the dike, raising the embankment in 12-inch layers, allowing the fill to drain and then suitably compacting the layer. In this method, excess water and fines will be wasted over a wasteway, and deposition of materials will be controlled by baffles. Bids will be obtained by placing the materials by either method and the most economical method used.

C. DIMENSIONS AND DESCRIPTIONS. - Sections of the embankment as designed are shown on Plates Nos. 15, 16 and 17. The elevation of the Connecticut River bed varies from Elevation 8.0 to Elevation 22.0 along the dike alinement. The top of dike grade varies from Elevation 40.7 to Elevation 39.1. The side slopes are 1 on 3 with both slopes spot sodded and seeded except where the riverside slope is riprapped. The cut-off trench will be carried to depths varying from 4 to 20 feet depending on material found upon excavation. Variations in trenches necessary for the different depths have been detailed on Plate No. 17. The toe drain varies in depth from 2 to 9 feet as shown on Plate No. 17. From Station 99 + 40 to the railroad embankment a temporary plug of random fill material is to be placed to prevent the river from flowing between the dike and the railroad embankment, during high water, and causing damaging scour. The top elevation of this plug will be slightly above the present railroad embankment so that overtopping will not occur until the elevation of the water on both sides is approximately equalized. This plug will be removed prior to the construction of the future extension of the dike.

D. HEIGHT AND TOP WIDTH OF DIKE. - The height of the dike has been determined as stated in Paragraph IV B. The treatment of the side slopes and the top width of 10 feet are designed to effectively protect the dike from damage by spray or wave action. These determinations satisfy Criterion 1 of Section VI.

E. STABILITY OF SLOPES. - The design of the embankment is based on experience and comparison with existing dikes and dams. The flat slopes, compaction of embankment and surface protection in conjunction with a rock filled toe drain at the landside toe all contributed to the stability of the slopes. These determinations satisfy Criterion 2 of Section VI.

1. Riprap Slope Protection. - High velocity flows in the river made it necessary to provide riprap protection for the slopes of the dike to prevent erosion. A study of the flows in the river, with allowance made for the new conditions and resulting higher velocities, indicated that the riprap was required in some places and could safely be omitted in other places. See Plate No. 14. No protection for the river banks was considered necessary except at the location of the concrete wall and other points of minor importance. The riprap will be hand-placed to a thickness of 12 inches and will rest on a 6-inch bed of gravel which is designed to prevent undercutting of the riprap.

2. Concrete Crib Wall. - Near Station 117 + 00 a situation developed where a considerable saving to the Town of East Hartford could be effected at a somewhat lesser cost to the United States by constructing a low concrete crib wall at the toe of the dike. The purpose of the cribbing is to foreshorten the normal toe and avoid the

necessity of removing several buildings. A 10-foot areaway was provided in which to construct the sewer outside the toe of the dike.

F. SATURATION LINE. - Considering the impervious quality of the blanket material and its favorable difference in permeability as compared with the body of the dike, a very low line of saturation is to be expected. Seepage through the foundation will not be excessive owing to the cut-off or to natural conditions. The rock filled toe drain will hold the discharge well within the landside toe. Where the dike is five feet high or greater, the drains are provided. See Plate No. 17. The drains are sloped so as to direct the seepage flow to the lateral drains and thence into the collector drain. See Section VIII Paragraph A 1. Sheet pile cut-offs are provided along two sections of the dike as described in Paragraph IX, H. 7.

The above determinations satisfy Criterion 3, 4, and 5, mentioned in Section VI.

G. ACCESS RAMPS. - Ramps are provided at strategic points in conformity with the needs of the Town of East Hartford. These crossings are necessary for fire protection and access to lands lying beyond the dikes. The ramps have a grade of 7% and top width of 10 feet. The road surfacing on the river-side ramp is usually macadamized to prevent erosion by river currents.

H. STABILITY OF FOUNDATION. - After thorough study of the geological investigations (Section III), seepage problems, and the proportions of the proposed dike, the foundation for the dike is considered sufficiently stable to resist any undue stresses caused by the embankment loads. This determination satisfies Criterion 8 in Section VI.

I. CUT-OFF. - The dike is provided throughout its length with a cut-off. Four general types are used depending on subsurface conditions. The four types are: (a) A steel sheet pile membrane with top embedded in the impervious part of the dike and the bottom impervious clay stratum lying at approximately elevation minus 20 m.s.l. (b) A deep backfilled trench excavated through pervious or doubtful material and about 5 feet into impervious soil, with a maximum depth of about 20 feet. (c) A shallow trench, usually 5 feet deep, keying the impervious part of the dike into the impervious foundation. This excavation may also be considered an exploration trench. (d) A horizontal impervious blanket at the river-side toe designed to reduce seepage. See Plates Nos. 15 and 16.

a. Type "A" Cut-off.- It was found necessary to use sheet piling between Stations 110 + 50 and 116 + 50 and between Stations 142 + 50 and 177 + 00. In the first-mentioned stretch the upper impervious stratum of the foundation has been removed by erosion or other agents exposing the layer of Class 2 and 4 sands which is open to the river. Between Stations 142 + 50 and 177 + 00 it was found necessary to use a sheet-pile cut-off as the natural blanket was found to be inadequate or lacking. A sheet-pile cut-off was used under the concrete wall.

b. Type "B" Cut-off. - Large quantities of fill have been placed on the foreshore between Station 106 and 135 (Plate No. 8) necessitating the use, in some places, of a deep trench cut-off or sheet piling. The trench was used where the natural blanket was found by subsurface explorations to be of adequate thickness. The bottom of the trench will be carried through the fill material and about 5 feet into the natural impervious layer. The trench will be backfilled with impervious fill, except that where the deepest trenches are used the impervious fill is limited to 7 feet in thickness in the interests of economy. See Plate No. 17.

c. Type "C" Cut-off. - At all points where the types "A", "B", or "D" cut-offs are not used a combination cut-off and exploration trench about 5 feet deep is called for. An exception to Paragraph b. above occurs in the vicinity of Connecticut Boulevard where the type "C" cut-off was used. The fill, at this point, is indicated by bore holes, and other investigations to consist of rocks, debris, and soils. The soils form almost the whole body of the fill and are of a highly impervious nature. Considering the small differential in the head, the great length of the path of percolation, and the impervious nature of the fill, the type "C" cut-off was found adequate.

d. Type "D" cut-off. - Commencing at Station 177 + 00 it was found that a satisfactory and economical cut-off could be obtained by the use of a blanket of impervious material extending laterally about 200 feet beyond the river-side toe of the dike. The blanket ranges in thickness from 5 to 3 feet and follows the undulating surface of the land. It is anticipated that when the dikes are completed a dead water area will occur in this general vicinity during times of flood and that fine materials will be deposited, thus bringing about an increasingly effective blanket each year.

J. REMOVAL OF OLD SEWER PIPES. - In the preparation of the foundation for the dike various sewer pipes will be encountered. These pipes will be removed and plugged as shown on Plate No. 22 entitled "Cherry Street Outfall."

X. STRUCTURAL DESIGN

X. STRUCTURAL DESIGN

A. GENERAL. - The reinforced concrete flood wall at the bulk oil terminal was designed to resist a principal load coming from the assumed head of water which is taken as about two feet higher than the wall and therefore somewhat greater than the comprehensive plan flood stage. The secondary loading is due to the difference in elevation of the ground surface on opposite sides of the wall. The top of the wall will be constructed to elevation 37.4 as discussed in Paragraph VIII A. This height of wall provides a freeboard of 3 feet above the recommended grade for design flood, based on the comprehensive plan of 20 reservoirs. To accommodate a higher river stage resulting from an anticipated modification of the design flood the wall was designed to withstand a head of water equal to 2 feet above the top of wall as constructed in accordance with instructions from the Chief of Engineers. The wall could be raised by a temporary breast work at times of an extreme major flood. See Paragraph C 1. Owing to the possibility of damage by impact from ice floes or barges in the river, the lateral support was placed on the landside to act principally in compression(as buttresses). The tendency of the structure to slide necessitated a deep lug or key on the trailing side of the base. The peculiar condition of tension plus bending at the throat of the lug made a haunch necessary at the base of the stem (vertical wall). Shearing blocks were used along the plane between buttresses and base slabs to resist the tendency to slide along that plane. The interaction of a tie rod, connecting the base of the concrete wall with the top of a timber bulkhead, was investigated. A valve-chamber box, located on the riverside of the wall, was designed as an integral part of the wall. The local surcharging of the soil by the oil tanks adjacent to the wall has been neglected, inasmuch as its action increases the stability of the structure. Likewise, the restraint offered by the sheet piling to both lateral and vertical forces has been neglected, its action tending to increase the factor of safety against overturning and sliding. On the landside of the wall a small fill or bank will be placed against the wall as fire protection. The fill on the riverside of the wall is protected from erosion by a layer of riprap. The sheet-pile cut-off at the ends of the flood wall are continued a distance up and downstream to eliminate the possibility of erosion and undercutting. A bulkhead door is to be built into the wall to provide pedestrian access to the landing from the plant. The plans provide no access for vehicles to the landing. At any time a roadway is desired, space may be found on the foreshore to join the landing with the ramp to the north. With the wall being so high it was deemed advisable to treat the riverside face architecturally. This ornamentation is in heavy relief and will be visible at a distance especially to river traffic which is largely pleasure craft.

B. SPECIFICATIONS FOR STRUCTURAL DESIGN.

1. General. - The structural design of the floodwall has been executed, in general, in accordance with standard practice. The specifications which follow cover the conditions affecting the design for stability and for reinforced concrete.

2. Unit Weights. - The following unit weights for materials were assumed in the design of the structure:

Water	62.5	pounds	per	cubic	foot
Dry earth	100	"	"	"	"
Saturated earth	125	"	"	"	"
Concrete	150	"	"	"	"

3. Earth Pressures. - In computing active earth pressures, equivalent fluid pressures computed by the use of Rankine's formula were used. They are as follows:

Earth, dry, equivalent liquid loading = 35 pounds per cubic foot.
Earth, saturated, equivalent liquid loading = 80 pounds per cubic foot.

In computing passive resistances, Rankine's formula was used with the coefficient of internal friction = 30 degrees.

4. Hydrostatic Uplift.

a. Riverside of Sheet Piling

Full head due to head water.

b. Landside of Sheet Piling

(1.) At landward toe, full head due to tailwater. See Paragraph C 1.

(2.) At sheet piling, full head due to tailwater plus one-half the difference between headwater and tailwater.

(3.) At intermediate points, the uplift is assumed to vary directly with the distance from the toe.

5. Overturning. - The resultant of all external loads, including hydrostatic uplift and excluding base pressure, shall fall within the middle third for all loading conditions.

6. Sliding. - The total horizontal forces due to external loads shall not exceed the resistance available from friction and passive resistance. The coefficient of friction to be used in such computations

is 0.45.

7. Bearing. - The total bearing pressure, equal to the sum of hydrostatic pressure plus the remaining effective base pressure, shall not exceed the maximum allowable base pressure of 4,000 pounds per square foot which was determined by the District Soils Laboratory.

8. Frost Cover. - All footing bases shall lie at least 4 feet beneath the surface of the ground to avoid heaving by frost action.

9. Path of Creep. - The path of creep criteria will not govern as the sheet piling will be driven into a stratum of impervious material.

10. Reinforcing Steel. - The steel assumed to be used is new billet steel, intermediate grade, deformed bars. The effective cross-sectional areas are taken as net, and the working stress used is as follows:

Tension, main steel = 18,000 pounds per square inch.

11. Reinforced Concrete. - In general, the design of the reinforced concrete was in accordance with the recommendations of the Joint Committee and the American Concrete Institute. Specifically, the working stresses are as follows:

a. Ultimate Strength. - The allowable working stresses in concrete are based on an average ultimate compressive strength of 3400 pounds per square inch in 28 days.

b. Flexure. - Extreme fiber stress in compression = 800 pounds per square inch. Fiber stress in bearing = 500 pounds per square inch. No allowance is made for the beneficial action of concrete in tension.

c. Shear.

Without special anchorage = 60 pounds per square inch
With special anchorage = 90 " " " "

d. Bond.

Without special anchorage = 100 pounds per square inch
With special anchorage = 200 " " " "

e. Embedment.

Minimum embedment to develop bond = 40 diameters

f. Ratio of Moduli of Elasticity.

$E_s/E_c = n = 12$

g. Protective Concrete Covering.

In lower face of wall base slab = 4 inches

In top face, stem, counterforts and valve chamber = 3 "

h. Temperature Steel.

Minimum steel in any exposed face is 5/8"Ø bars spaced one foot on centers.

12. Wrought Iron. - The allowable working stress of wrought iron is taken as

Tension (intermittent use) = 16,000 pounds per square inch.

13. Structural Steel. - The design of steel structures has been governed by the "Standard Specifications for Steel Construction", of the American Institute of Steel Construction.

C. BASIC ASSUMPTIONS FOR DESIGN

1. Loadings. - The assumed water elevation is 39.4 feet. The average elevation of the toe drain as used in the computations is 2.0 feet above top of base slab. The elevation of the earth fill varies with the stationing along the river. Only one case for stability of the

structure needs to be considered, that of maximum flood stage in the river, as the secondary loading due to differences in earth elevation is negligible in comparison.

2. Structural Action. - The structure is assumed to act as a counterfort wall, resisting lateral forces in both directions. The restraint of the sheet piling is neglected.

3. Wall Stem. - The stem is designed to carry the differential load due to water pressure and earth pressure by beam action to the counterforts. The effect of continuity and restraint is represented by an arbitrary increase of the maximum moment to $wL^2/10$. Identical steel is placed in both faces to conform to accepted practice, and this steel is assumed to carry the minor loads.

4. Base Slab.

a. Landside. - The load is assumed to be carried by beam action to the counterforts. The beams are assumed fixed at the counterforts, and the maximum moment is $wL^2/12$. Identical steel is placed in both faces.

b. Riverside. - The critical section of the throat across the lug is designed to resist tension and bending. In addition to the earth pressures and weights, the friction is assumed distributed uniformly (vertically) over the lug (Tentative Standards of Design of Keys by Office of Chief of Engineers).

5. Counterforts. - The counterforts are designed to resist the compressional, tensional, and shearing forces induced by the slabs.

XI. CONSTRUCTION PROCEDURE

XI. CONSTRUCTION PROCEDURE.

A. **SEQUENCE OF OPERATIONS.** - Assuming that the contract for the work will be let on or before May 20, 1939, and construction will commence on June 1, it is contemplated that the work will be completed by June 30, 1940. The divisions of working seasons being as follows:

First construction season - May 1939 to December 1939.

Second construction season - March 1940 to July 1940.

First Construction Season. - The work to be accomplished during this season will be (1) preparation of site; (2) completion of stripping; (3) completion of all excavation except in the borrow pits; (4) placing of a portion of the embankment; (5) driving of all sheet piling; (6) placing of a portion of the riprapping; (7) completion of the outfalls and drainage systems; and (8) the construction of the concrete flood wall.

Second Construction Season. - As soon as the weather and river conditions permit, the work required to complete the dikes shall be resumed. All work under this contract will be completed during this period.

The proposed time limit of operations is based upon a possible construction schedule which is tabulated below.

TABLE NO. 6
EAST HARTFORD DIKE
STA. 98+00 to STA. 170+00
CONSTRUCTION PROGRAM

Item	From	To	No. of Working Days	Quantity	Daily Rate
First Season					
Preparation of Site	June 1	Sept. 1	60	8 acres	0.13 acre
Stripping	July 1	Oct. 1	60	12,300 cu.yds.	205 cu.yds.

Item	From	To	No. of Working Days	Quantity	Daily Rate
First Season (Cont.)					
Excavation	June 1	Oct. 1	80	33,200 cu. yds.	415 cu.yds.
Embankment (pervious)	July 1	Dec. 1	100	203,000 cu. yds.	2,030 cu.yds.
" (impervious)	July 1	Dec. 1	100	80,000 cu. yds.	800 cu.yds.
Sodding and seeding	Aug. 1	Oct. 1	40	4 acres	0.1 acre
Sheet piling	July 1	Nov. 1	80	125,000 sq.ft.	1,560 sq.ft.
Riprap	Aug. 1	Dec. 1	100	8,500 cu. yds.	85 cu.yds.
Outfalls	July 1	Sept. 1	40	-	-
Drainage systems	July 1	Nov. 1	80	-	-
Concrete flood wall	June 1	Dec. 1	120	5,000 cu. yds.	42 cu.yds.
Second Season					
Embankment (pervious)	April 1	June 30	60	100,000 cu. yds.	1,330 cu.yds.
" (impervious)	April 1	June 30	60	36,000 cu. yds.	600 cu.yds.
Sodding and seeding	April 1	June 30	60	8 acres	0.13 acre
Riprap	March 1	June 1	60	3,000 cu.yds.	50 cu.yds.

Contract to be let on or before May 20, 1939.

Completion of work: June 30, 1940.

Number of calendar days for construction: 400.

1. Preparation of Site. - The work involved is (1) clearing; (2) grubbing; (3) removal of masonry foundations and structures. Clearing consists of the removal of trees, brush and shrubs, and removal of buildings under the dike site. Grubbing consists of the removal of stumps, roots and vegetable growth over the area cleared and under the site of the dike. Removal of any cellar foundations, culverts or other small

structures will then be accomplished.

2. Stripping. - When the site is cleared and grubbed, all undesirable material under the dike site is then removed by stripping.

3. Excavation. - While the site is being cleared, grubbed and stripped, excavations for the foundation of the concrete flood wall may be accomplished. Upon completion of the stripping, the toe trenches may be excavated, one for drains and the other for exploration and cut-off. Material from these excavations may be used in the random fill section of the embankment.

4. Borrow Excavation. - Borrow excavation will start soon after the toe trenches are commenced. This will consist of excavating and hauling suitable impervious material for the impervious portions of the dike and dredging from the river suitable pervious material for the dike and ramps. The pervious material may be dredged directly into place or stock-piled before placing, depending on the more economical method as determined from bid prices.

5. Steel Sheet Piling. - The steel sheet piling will be driven in those portions of the dike where it is required before placement of fill is started. Under the concrete flood wall it will be driven before concrete is placed.

6. Embankment Construction. - As soon as the foundation and toe trenches have been prepared and the toe drain constructed, the embankment construction will take place. The embankment may be constructed entirely by the rolled fill method or as much of the pervious portion of the embankment as practicable constructed by the hydraulic fill method and the remainder by the rolled fill method at the option of the contract-

ing officer. The pervious fill will be obtained from Borrow Areas "A1," "A2" and "A3." The impervious fill will be obtained from Borrow Areas "C" and "E." The random fill will come from excavations on the site provided the material is suitable. Rock for the hand-placed riprap may be obtained from nearby quarries. It is expected that the rock will be placed simultaneously with the fill. Use of modern construction equipment and standard methods of construction are contemplated throughout. The rolled fill will be placed by truck or crawler wagons and rolled by sheep's-foot, cylindrical, or disc rollers, as may be required. Pneumatic hand tampers will be used in corners or near structures. As the embankment is to be built in two seasons, it will be desirable to construct those portions that will present the least tendency to scour and provide the best measure of protection from the flood waters.

7. Drainage System and Outfalls. - The construction of the drainage systems should start as soon as practicable. It will be necessary to build the outfalls first so that no delay will be caused to the construction of the dike proper. Excavation, pipe laying, and backfill may be completed before the manholes and valve chambers are finished. These systems shall be completed during the first construction season.

8. Flood Wall. - Excavation for the flood wall will be started at the beginning of the first working season. The construction of the wall and related structures will then be started and shall be finished by the end of the first working season.

9. Miscellaneous. - The placing of sod and riprap will keep pace with the construction of the dike.

The job will be cleaned up as soon as practicable.

B. <u>LABORATORY AND FIELD TESTS DURING CONSTRUCTION</u>. - The Soils Laboratory at Providence, Rhode Island, and the field soils laboratory at East Hartford, Connecticut, will perform certain tests as necessary to investigate and record the characteristics of various types of soil used in construction.

1. <u>Field Laboratory tests.</u>

<u>a.</u> <u>Mechanical Analysis and Classification of Soils</u>. - Results will indicate the type of material placed in the various sections of the embankment, and will serve as a basis of confirmation of the visual inspection of the materials made before placement.

<u>b.</u> <u>Determination of Water Content and Density in Place</u>. - Results, when compared with results of laboratory tests performed previous to the beginning of construction may be used in establishing the most advantageous procedures for placement and compaction. Preliminary test results have been obtained for two distinct types of materials:

(1) The relations between water content and compacted density have been determined by the Proctor method of laboratory analysis for those materials whose density and cohesion are affected by water content during compaction.

(2) The lowest and highest densities at which the soil could be placed in the laboratory have been determined for the free-draining, cohesionless materials. The lowest density was obtained by pouring the soil into a container, with a minimum of vibration, and no tamping. The highest density was obtained by placing the soil in a standard Proctor cylinder in one inch layers, each layer being tamped

vigorously subsequent to placing.

Oven-dried material was used in both instances.

The results of field laboratory tests on samples of the first type of material may be compared directly with the optimum water content and density at that optimum water content previously established by the Proctor test. The comparison of results of tests on samples of the second type of material with previous tests may be expressed as follows:

$$P_v = \frac{e_0 - e}{e_0 - e_{100}} \times 100$$

in which P_v = degree of compaction, %

e = void ratio of material in embankment

e_0 = void ratio at minimum laboratory compaction

e_{100} = void ratio at maximum laboratory compaction

c. Compaction Characteristics of Borrow Materials. - Tests will be performed at intervals, following methods outlined under b, to confirm the results of preliminary tests, and to obtain additional data as required.

2. Soils Laboratory Tests. - Supplementary shear and permeability tests will be performed at the Soils Laboratory at Providence, Rhode Island, for the purpose of obtaining additional information and to confirm results obtained previous to the beginning of construction.

3. Classes of Materials. - The embankment will be made up of various types of fill:

a. Pervious fill.

b. Impervious fill.

c. Random fill.

d. Gravel bedding.

e. Riprap.

f. Crushed rock.

a. Pervious Fill. - Pervious material for rolled fill or hydraulic fill, at the option of the contracting officer, will be used in the construction of the main portion of the embankment. Material will be obtained from Borrow Areas "A1," "A2" and "A3." The fill will be placed so that the coarser portions will be near the landside part of the dike. If hydraulic fill is transported direct to dike, special precautions will be taken to prevent an excess of fines from going into the embankment. The fill will be periodically tested to insure that the proper distribution and compaction of materials are obtained.

b. Impervious Fill. - Fill for the impervious portion of the embankment and for the blanket will be obtained from Borrow Areas "C" and "E". Tests will be made at frequent intervals to secure and maintain the maximum density of material.

c. Random Fill. - Suitable materials will be obtained from excavations on the site. These fills will be placed inside of and adjacent to the impervious portion of the dike, where the nature of the material will least affect the stability of the section.

d. Gravel Bedding. - The gravel bedding for the hand-placed riprap and rock filled toe drain will be obtained from nearby commercial sources where it will be screened. No rigid laboratory tests will be required for this class of material in general, as visual inspection is considered sufficient.

e. Hand-placed Riprap. - The hand-placed riprap is placed on the riverside of the embankment. Rock may be obtained from nearby quarries.

f. Crushed Rock. - Crushed rock is placed in the rock filled toe drain. It may be obtained from nearby commercial sources.

C. PLACEMENT OF MATERIALS. -

1. Method of Placing. - The foundation will be grubbed, stripped, and plowed in preparation for placing the fill. The cut-off trench will be excavated and filled with impervious material placed and compacted in layers. The toe drain will be excavated and filled with gravel and crushed rock. The outfalls will be excavated, pipes laid, and backfilled. Except where the pervious portion is placed hydraulically the pervious, random, and impervious material will be brought up together and rolled in layers.

a. Wetting. - When material is placed in the dike it will be wetted to the proper degree so that maximum compaction will be obtained. Should the fill be too wet it will be spread in thin layers and be permitted to dry until the proper moisture content is obtained. Fill in place in the embankment that has become dry and cracked will be loosened and dampened before the succeeding material is spread.

b. Spreading. - The materials placed in the rolled embankment will be "bull-dozed" or otherwise spread in the required layers. Grading of material obtained from two or more borrow areas will be accomplished during the spreading. If the preceding layer of fill has become too hard or smooth to insure proper bond, it will be loosened by harrowing before the succeeding fill is placed.

c. Compacting. - When the moisture content and condition of the spread material is determined to be correct it will be rolled by suitable rollers such as a sheep's-foot tamper type for the impervious fill and a plain cylindrical or disc roller for the pervious fill. The rollers will be pulled by a crawler-type tractor which will supplement the rollers in compacting the material.

d. Stock-piling. - Except where pervious material is placed directly in the dike, it will be stock-piled and allowed to drain before being placed in the dike and rolled.

e. Hydraulic Placing of Fill. - As an alternate to stock-piling and rehandling, the materials for the pervious portion of the dike may be dredged directly into place. Material thus dredged into place will be placed in layers approximately 1 foot thick and compacted. The pervious portions will be constructed by hydraulic placement to the greatest height that is feasible, and then completed by dry methods.

D. FIELD CONTROL DURING CONSTRUCTION.

1. Field Control Necessary. - Close control of the material placed in the embankment will be maintained. It will be frequently tested to determine its suitability for the purpose intended and that the desired results are being obtained. It will be required that the equipment used in the construction of the embankment be maintained in good operating condition at all times.

a. Method of Testing. - Embankment materials will be sampled and tested at frequent intervals both before and after placing. The standard methods of testing will be employed in classifying the materials, and determining water content of rolled fill, compacted weights and compaction characteristics. These tests will be conducted by the field laboratory.

E. CONCRETE CONSTRUCTION.

1. Laboratory Control. - Materials used in the making of concrete will be tested at the Central Concrete Testing Laboratory, West Point, New York. Tests made will include the suitability of the aggregate and the compressive strength of concrete. The work of this laboratory will be supplemented by a small field laboratory set up at the site. The field laboratory will be used principally to control the quality of concrete during construction. Facilities will be available for grading the aggregates, designing mixes, making of slump tests, and for casting and curing concrete cylinders for compression strength tests. Samples of the aggregates to be used will be submitted for testing at least 30 days in advance of placing.

a. Cement. -Cement will be tested by a recognized testing laboratory and results of these tests will be known before the cement is used. Portland cement of a well-known and acceptable brand will be required throughout.

b. Fine Aggregate. - Natural sand will be used as fine aggregate and may be obtained from approved commercial sources. The aggregate will be subjected to careful, thorough analysis, including magnesium sulphate soundness tests, and tests made on mortar specimens for compressive strength.

c. Coarse Aggregate. - Washed gravel or crushed stone of required sizes obtained from approved commercial sources will be used as coarse aggregate. It will consist of hard, tough and durable particles free from adherent coating and vegetable matter. Only a small amount of soft, friable, thin or elongated particles will be allowed. The aggregate will be subjected to freezing and thawing tests and to careful, thorough analysis, including magnesium sulphate test for soundness.

d. Water. - The amount of water used for each batch of concrete will be predetermined; in general, it will be the minimum amount necessary to produce a plastic mixture of the strength specified. Slump tests will be required in accordance with specifications. The water used will be fresh, clean and free from injurious amounts of oil, acid, alkali or organic matter.

2. Field Control.

a. Storage. - The concrete components will be stored separately before mixing. The cement will be stored in a thoroughly dry, weather-tight and properly ventilated building, with adequate provisions for prevention of the absorption of moisture. The fine and coarse aggregates will be stored in such a manner that inclusion of foreign material will be avoided.

b. Mixing. - The exact proportions of all materials in the concrete will be predetermined. The mixing will be done in approved mechanical mixers of a rotating drum type, with adequate facilities for accurate measurement and control of each of the materials used. The size of batches will be as directed. Samples will be taken for slump tests and compressive strength tests. Government inspectors will at all times supervise and inspect the mixing procedure.

c. Placing. - Concrete will be placed as soon as possible after mixing and before initial set has occurred. All concrete will be placed upon clean, damp surfaces. Mass concrete will be placed monolithic. Mechanical vibrators will be used, and forking or hand-spading will be required adjacent to forms on exposed faces in order to insure smooth, even surfaces. Locations of vertical and horizontal construction joints as well as contraction and expansion joints are indicated on the drawings. The locations of construction joints are tentative only and may be changed to suit conditions in the field. Before placing the concrete, all reinforcing steel will be inspected and pouring will be supervised and directed by Government inspectors. Adequate precautions will be taken if concrete is to be placed in cold or hot weather.

F. STRUCTURAL STEEL CONSTRUCTION.

1. Flood Wall Bulkhead Door. - A 3'-6" x 7'-0" door capable of withstanding the maximum head of water on the wall (23 feet) will be provided in the concrete wall at Station 147+30.68. It will be constructed of structural steel plate reinforced with horizontal angles. The gate will swing from hinges fastened to the upstream side of the gate frame which will be made up of structural angles securely embedded in the concrete. Swing bolts will be provided on the gate to draw it tight against the gate frame and reduce leakage to a minimum. As protection against damage by floating ice and debris, the door is recessed so as to be flush with the face of the wall.

XII. SUMMARY OF COSTS

XII. SUMMARY OF COSTS

The total construction cost of the East Hartford Dike from Station 98+00 to Station 125+50 and from Station 130+70 to Station 170+00 has been estimated at $786,000 including 10 per cent for contingencies and 15 per cent for engineering and overhead. This summary does not include the costs of any pumping stations nor the dike construction between Stations 170+00 and 186+10. These costs have been distributed as follows:

a.	Embankment	$336,100
b.	Concrete features	144,600
c.	Drainage systems	44,900
d.	Steel sheet piling	158,200
e.	Riprap, hand placed	36,600
f.	Miscellaneous	15,600

a. The embankment item consists of all excavation and fill for the dike with cut-off trench and toe drains, gravel bedding, topsoil, sodding and seeding, bituminous macadam road surfacing, concrete cribbing and ramps.

b. The concrete features consist of all excavation and backfill, cement, concrete, reinforcing steel, structural steel, copper water stops and alterations to existing bulkhead anchors involved.

c. The drainage systems consist of all excavation and backfill, tile pipe, reinforced concrete pipe, cast-iron pipe, gates, valves, care of sewage and water during construction and all brick manholes involved.

d. The sheet piling includes furnishing, placing and driving the piles.

e. The hand-placed riprap consists of gravel bedding, furnishing and placing the rock for riprap.

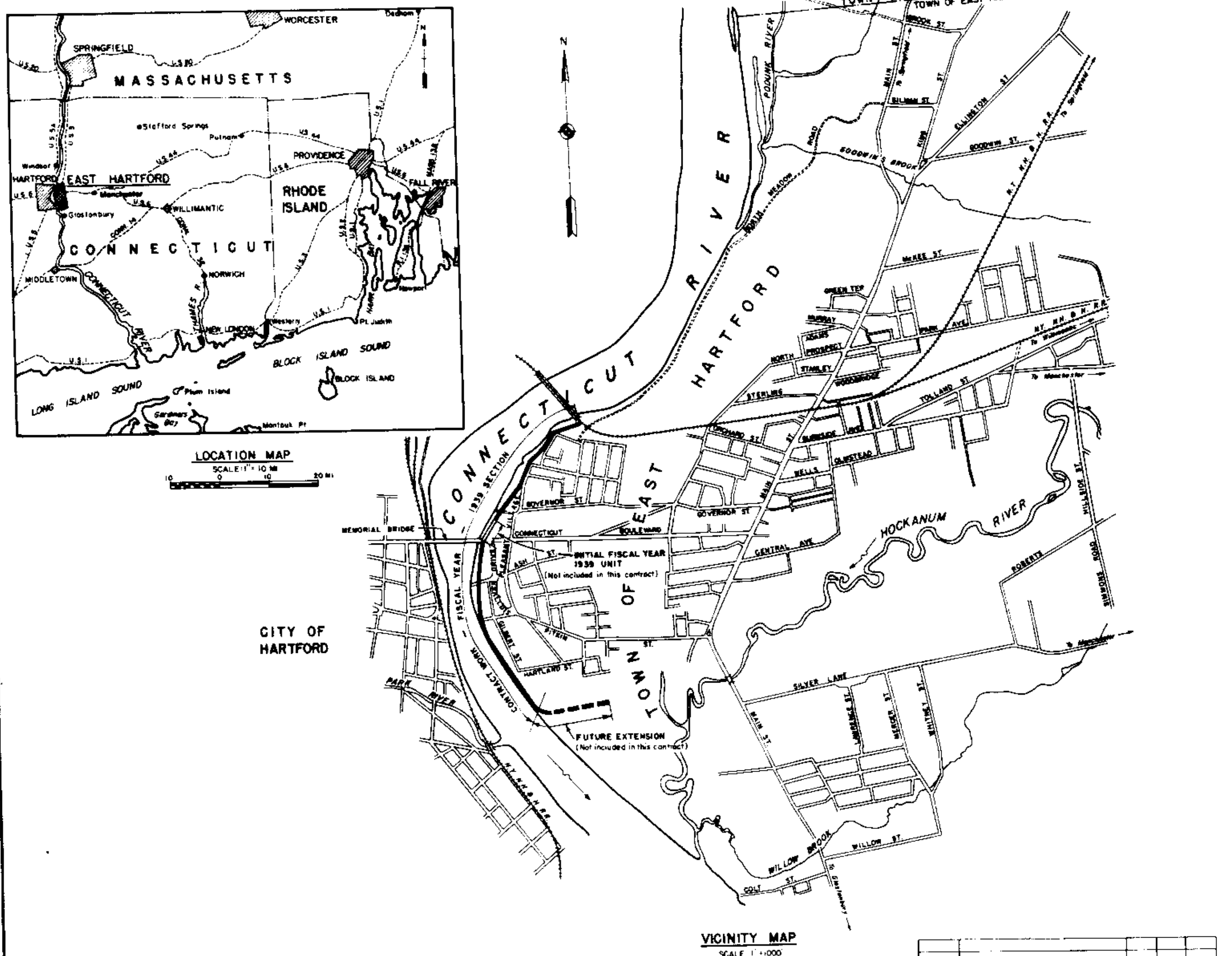
f. Miscellaneous items include preparation of site, miscellaneous fill and all other items not included under above Items a, b, c, d, and e.

XIII. PLATES

XIII. LIST OF PLATES

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



DATE	REVISION	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

PROJECT LOCATION AND INDEX

CONNECTICUT RIVER — CONNECTICUT

IN 72 SHEETS — SCALE: 1 IN.=1000 FT — SHEET NO

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. MARCH 1939

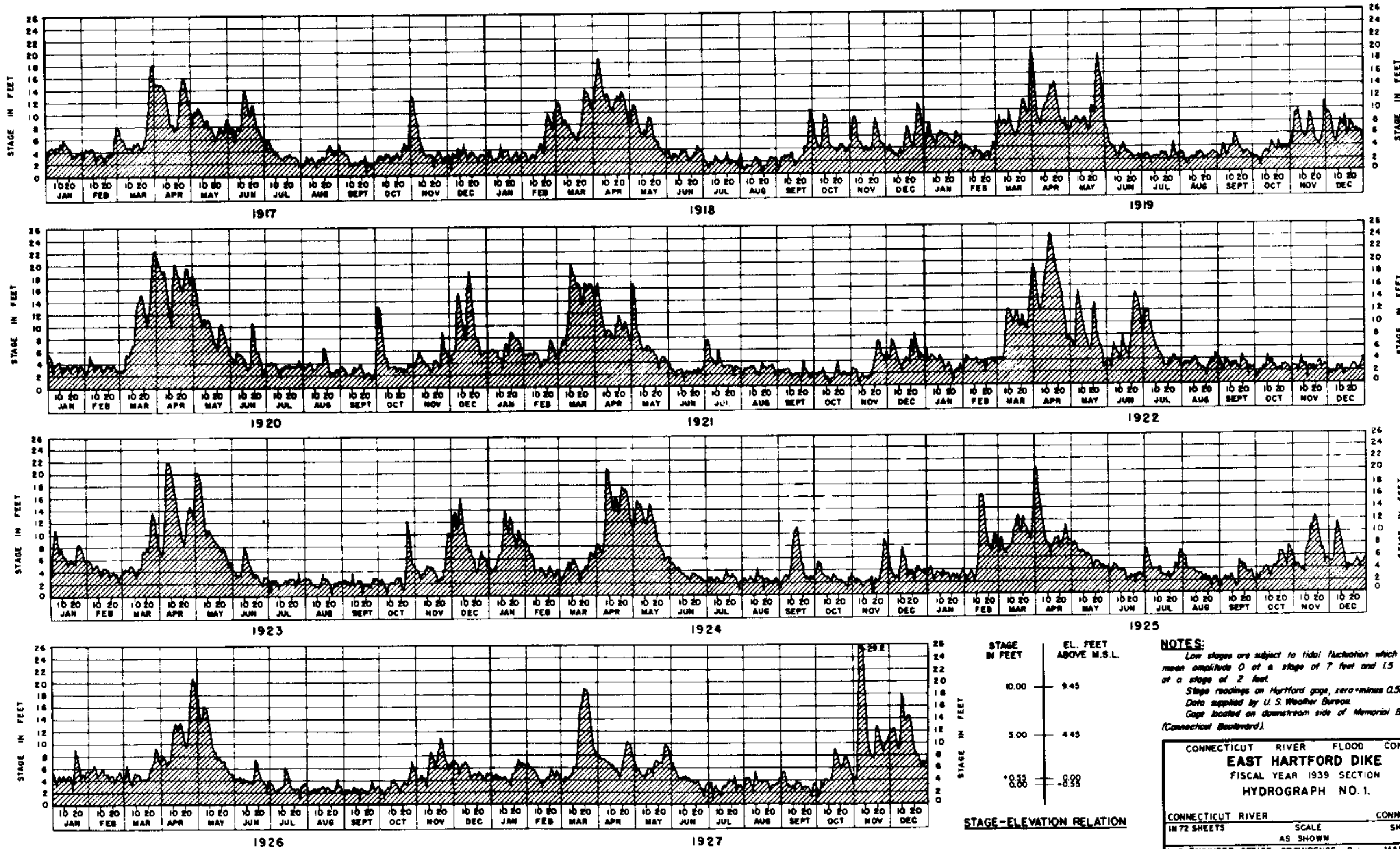
SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF F.C. ENGINEERING DIV. — APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: ASSOC. ENGINEER — DRAWN BY C.R.M. — TRACED BY R.H.B. — CHECKED BY

FILE NO. CT-4-312

PLATE NO. 1

(1)



NOTES:

Low stages are subject to tidal fluctuation which has a mean amplitude 0 at a stage of 7 feet and 1.5 feet at a stage of 2 feet.

Stage readings on Hartford gage, zero=minus 0.55' M.S.L.

Data supplied by U.S. Weather Bureau.

Gage located on downstream side of Memorial Bridge (Connecticut Boulevard).

DATE	REVISION	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

HYDROGRAPH NO. 1.

CONNECTICUT RIVER — CONNECTICUT

IN 72 SHEETS — SCALE AS SHOWN — SHEET NO. 2

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939

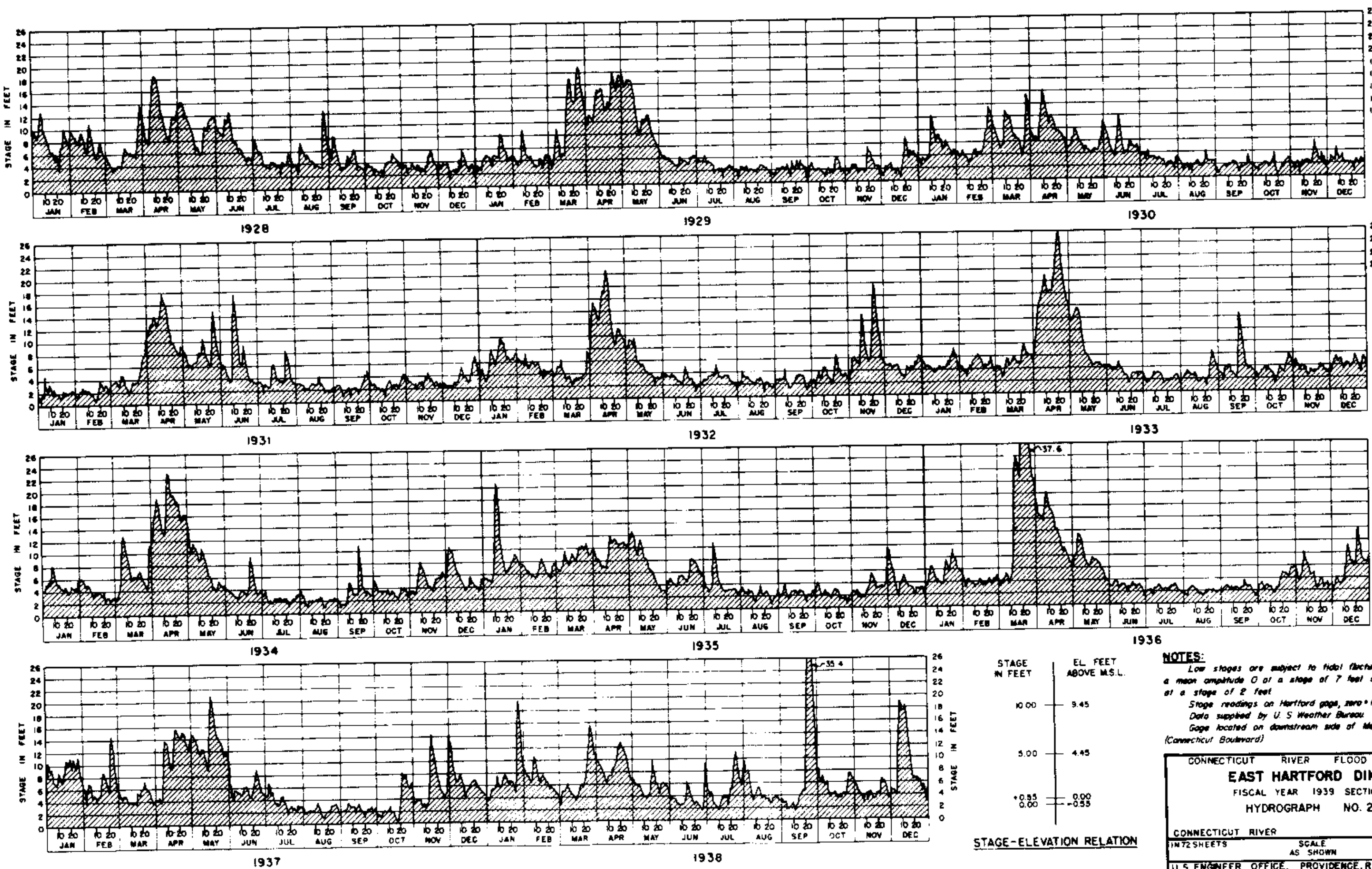
SUBMITTED: [signature] ENGINEER, HEAD, HYDRAULICS SECTION — APPROVAL RECOMMENDED: [signature] PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV. — APPROVED: [signature] DISTRICT ENGINEER

COMPILED: [signature] ASSOC. ENGINEER — DRAWN BY: W.W.V. & F.C.T. TRACED BY: P.J.G. CHECKED BY: W.C.M. — FILE NO. CT-3-1091

PLATE NO. 2

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



STAGE-ELEVATION RELATION

NOTES:

Low stages are subject to tidal fluctuation which has a mean amplitude 0 at a stage of 7 feet and 1.5 feet at a stage of 2 feet

Stage readings on Hartford gage, zero = minus 0.55' M.S.L.

Data supplied by U. S Weather Bureau

Gage located on downstream side of Memorial Bridge (Connecticut Boulevard)

DATE	REVISION	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

HYDROGRAPH NO. 2

CONNECTICUT RIVER — CONNECTICUT

IN 72 SHEETS — SCALE AS SHOWN — SHEET NO. 3

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939

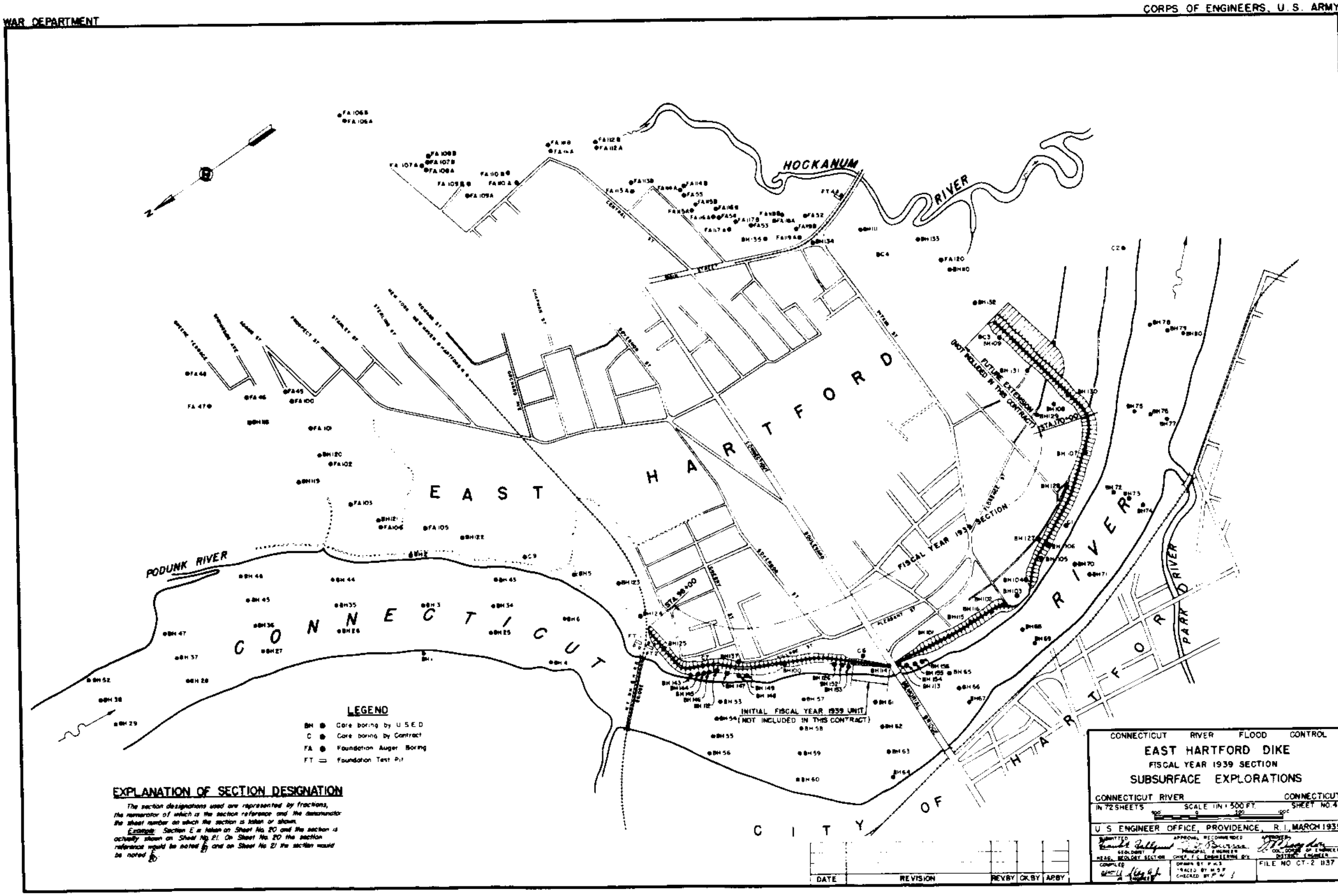
SUBMITTED: ENGINEER, HEAD, HYDRAULICS SECTION — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV. — APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: ENGINEER — DRAWN BY: W.H.V. & F.C.T. TRACED BY: D.H.B. CHECKED BY: — FILE NO. CT-3-1092

PLATE NO. 3

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



LEGEND

BH ● Core boring by U.S.E.D.
C ● Core boring by Contract
FA ● Foundation Auger Boring
FT ▭ Foundation Test Pit

EXPLANATION OF SECTION DESIGNATION

The section designations used are represented by fractions, the numerator of which is the section reference and the denominator the sheet number on which the section is taken or shown.

Example: Section E is taken on Sheet No. 20 and the section is actually shown on Sheet No. 21. On Sheet No. 20 the section reference would be noted $\frac{E}{21}$ and on Sheet No. 21 the section would be noted $\frac{E}{20}$.

DATE	REVISION	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

SUBSURFACE EXPLORATIONS

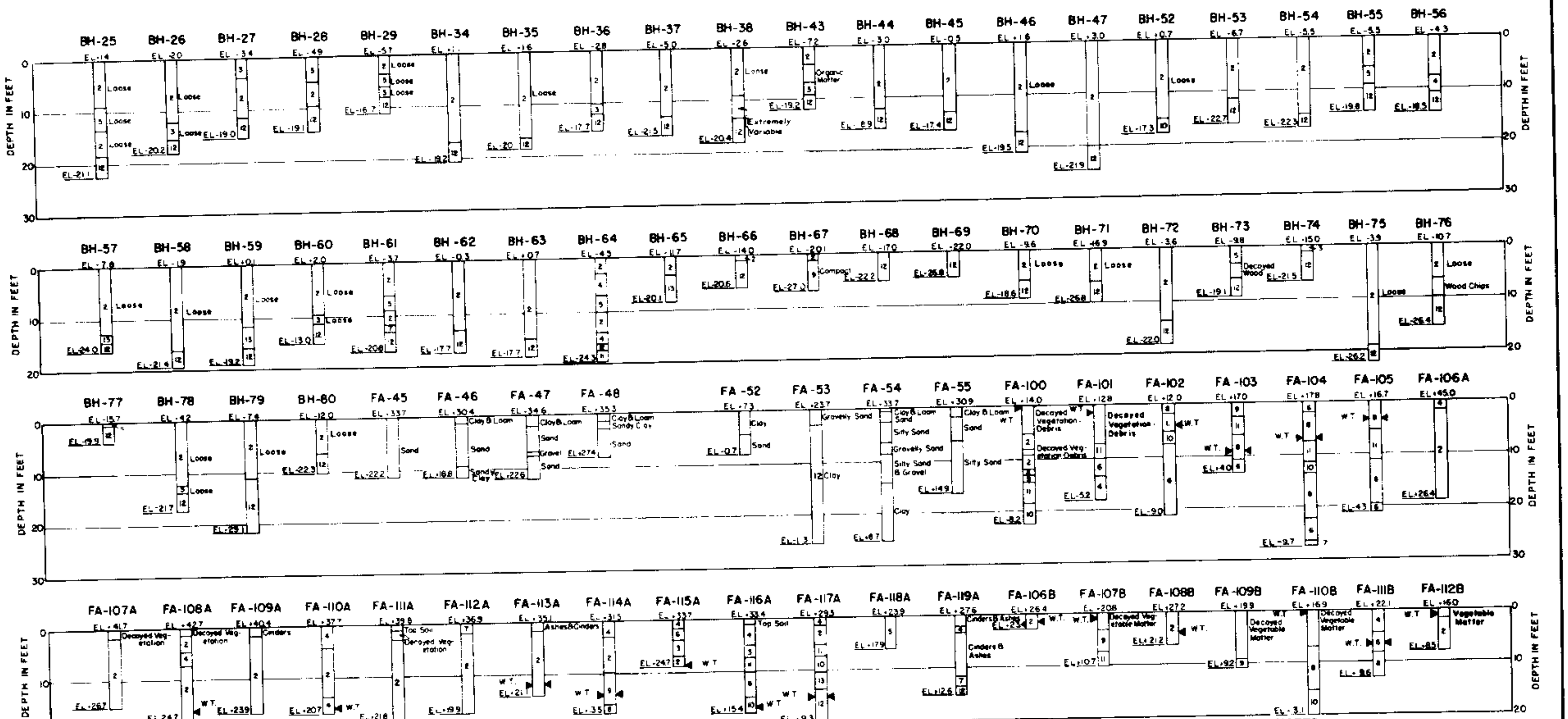
CONNECTICUT RIVER — CONNECTICUT
IN 72 SHEETS — SCALE 1 IN = 500 FT. — SHEET NO. 4

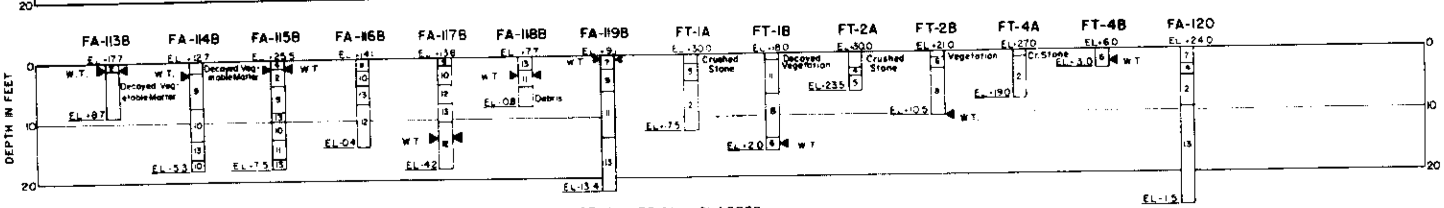
U S ENGINEER OFFICE, PROVIDENCE, R. I., MARCH 1939

SUBMITTED: GEOLOGIST, HEAD, GEOLOGY SECTION — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV. — APPROVED: COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED — DRAWN BY P.H.S. TRACED BY M.S.F. CHECKED BY P.W.J. — FILE NO CT-2 1137

PLATE NO. 4





NOTES

Elevations refer to M.S.L. datum.
For location of explorations see Sheet No 4.
For description of numerical classes see below.
Where exploration numbers are suffixed "A" or "B", "A" is located near the top of the bank and "B" near bottom.
Water table at time of exploration indicated thus: W.T.

DESCRIPTION OF NUMERICAL CLASSES

1. Clean Gravel - Contains little coarse to medium sand
2. Uniform Coarse to Medium Sand - Contains little gravel and fine sand
3. Variable - Graded from Gravel to Medium Sand - Contains little fine sand
4. Uniform Medium to Fine Sand - Contains little coarse sand and coarse silt
5. Variable Graded from Gravel to Fine Sand - Contains little coarse silt
6. Uniform Fine Sand to Coarse Silt - Contains little medium sand and medium silt
7. Variable Graded from Gravel to Coarse Silt - Contains little medium silt
8. Uniform Coarse to Medium Silt - Contains little fine sand and fine silt
9. Variable - Graded from Gravel to Medium Silt - Contains little fine silt
10. Uniform Medium to Fine Silt - Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt
10C. Uniform Medium Silt to Coarse Clay - Contains little coarse silt and medium clay. Possesses behavior characteristics of clay
11. Variable - Graded from Gravel or Coarse Sand to Fine Silt - Contains little coarse clay
12. Uniform Fine Silt to Medium Clay - Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt
12C. Uniform Clay - Contains little silt. Possesses behavior characteristics of clay
13. Variable - Graded from Coarse Sand to Clay - Contains little fine clay (colloids). Possesses behavior characteristics of silt
13C. Variable Clay - Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay

DATE	REVISION	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

RECORD OF SUBSURFACE EXPLORATIONS NO. 1

CONNECTICUT RIVER — CONNECTICUT

IN 72 SHEETS — VERT. SCALE 1 IN = 10 FT. — SHEET NO 5

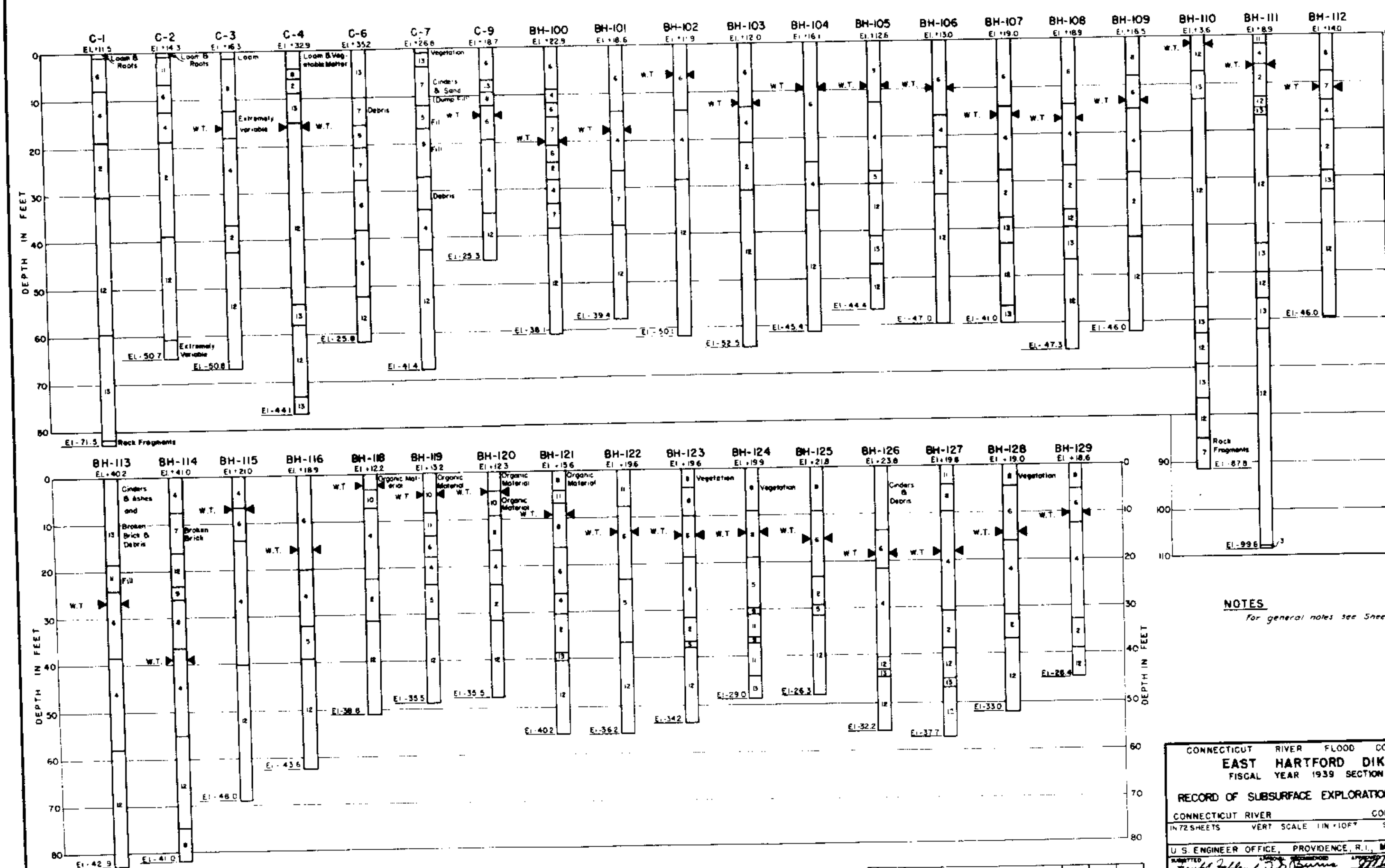
U. S. ENGINEER OFFICE, PROVIDENCE, R. I., MARCH 1939

SUBMITTED: [signature] GEOLOGIST, HEAD, GEOLOGY SECTION
APPROVAL RECOMMENDED: [signature] PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV.
APPROVED: [signature] LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: [signature] JR. ENGINEER
DRAWN BY R.H.B.
TRACED BY R.S.S.
CHECKED BY R.H.B.

FILE NO CT-2-1138

WAR DEPARTMENT



NOTES

For general notes see Sheet No. 5.

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

RECORD OF SUBSURFACE EXPLORATIONS NO. 2

CONNECTICUT RIVER CONNECTICUT

IN 72 SHEETS VERT. SCALE 1 IN = 10 FT. SHEET NO. 6

U. S. ENGINEER OFFICE, PROVIDENCE, R. I., MARCH 1939

SUBMITTED: GEOLOGIST, HEAD, GEOLOGY SECTION

RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIVISION

APPROVED: COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: JR. ENGINEER

DRAWN BY R.H.S. TRACED BY R.E.H. CHECKED BY P.H.L.

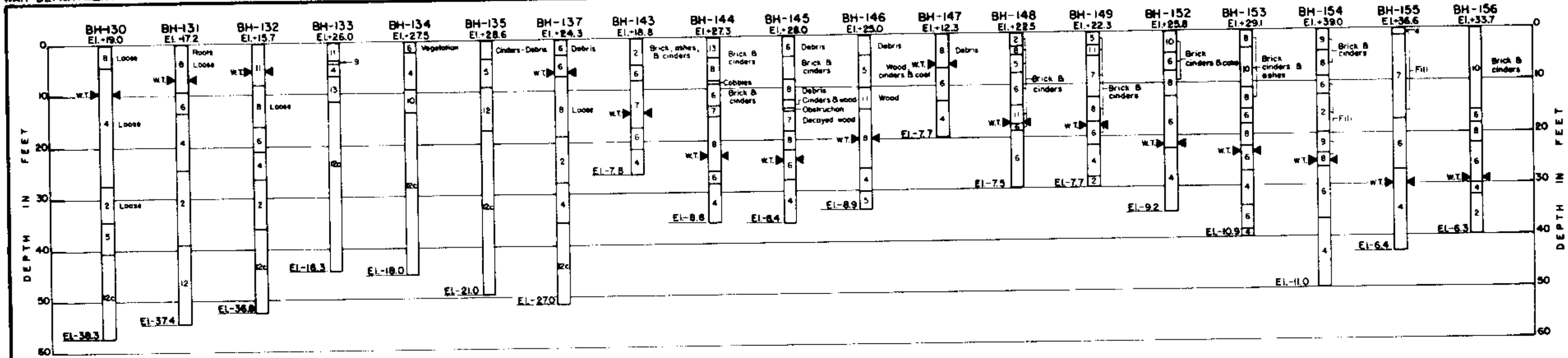
FILE NO. CT-2-1139

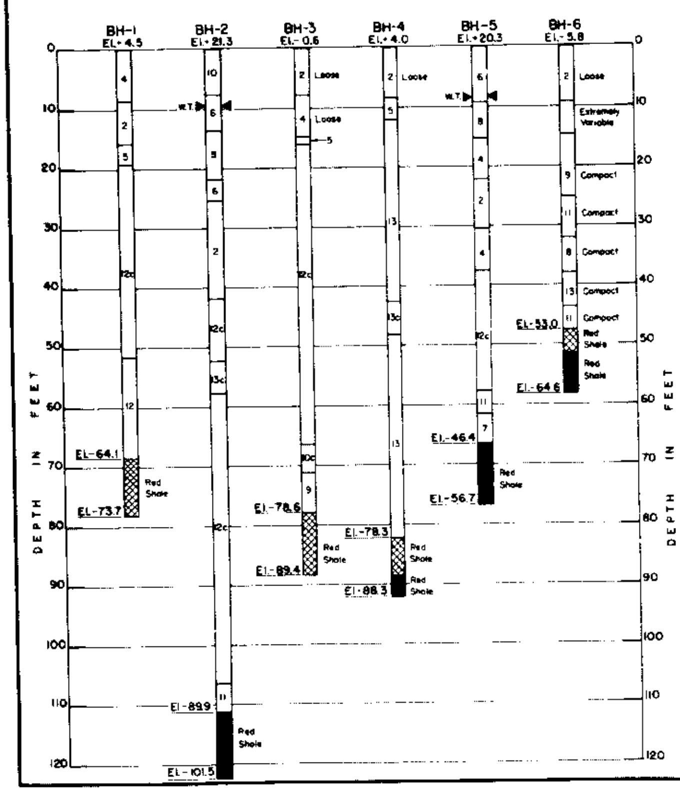
DATE	REVISION	REV. BY	CK. BY	AP. BY

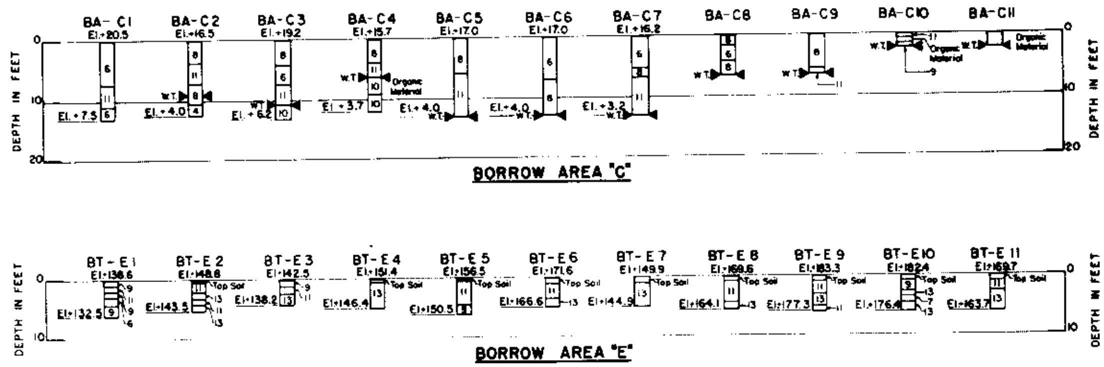
6

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY







LEGEND

W.T. *Water table at time of exploration.*

Soft or fractured bedrock.

Solid bedrock.

NOTES

Elevations refer to M.S.L. datum.
For description of numerical classes see Sheet No. 5
For locations of bore holes (BH) see Sheet No. 4
For locations of test pits (BT) see Sheet No. 9
For locations of auger borings (BA) see Sheet No. 9

DATE	REVISION	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

RECORD OF SUBSURFACE EXPLORATIONS NO. 3

CONNECTICUT RIVER, CONNECTICUT

IN 72 SHEETS VERT. SCALE: 1 IN. = 10 FT. SHEET NO. 7

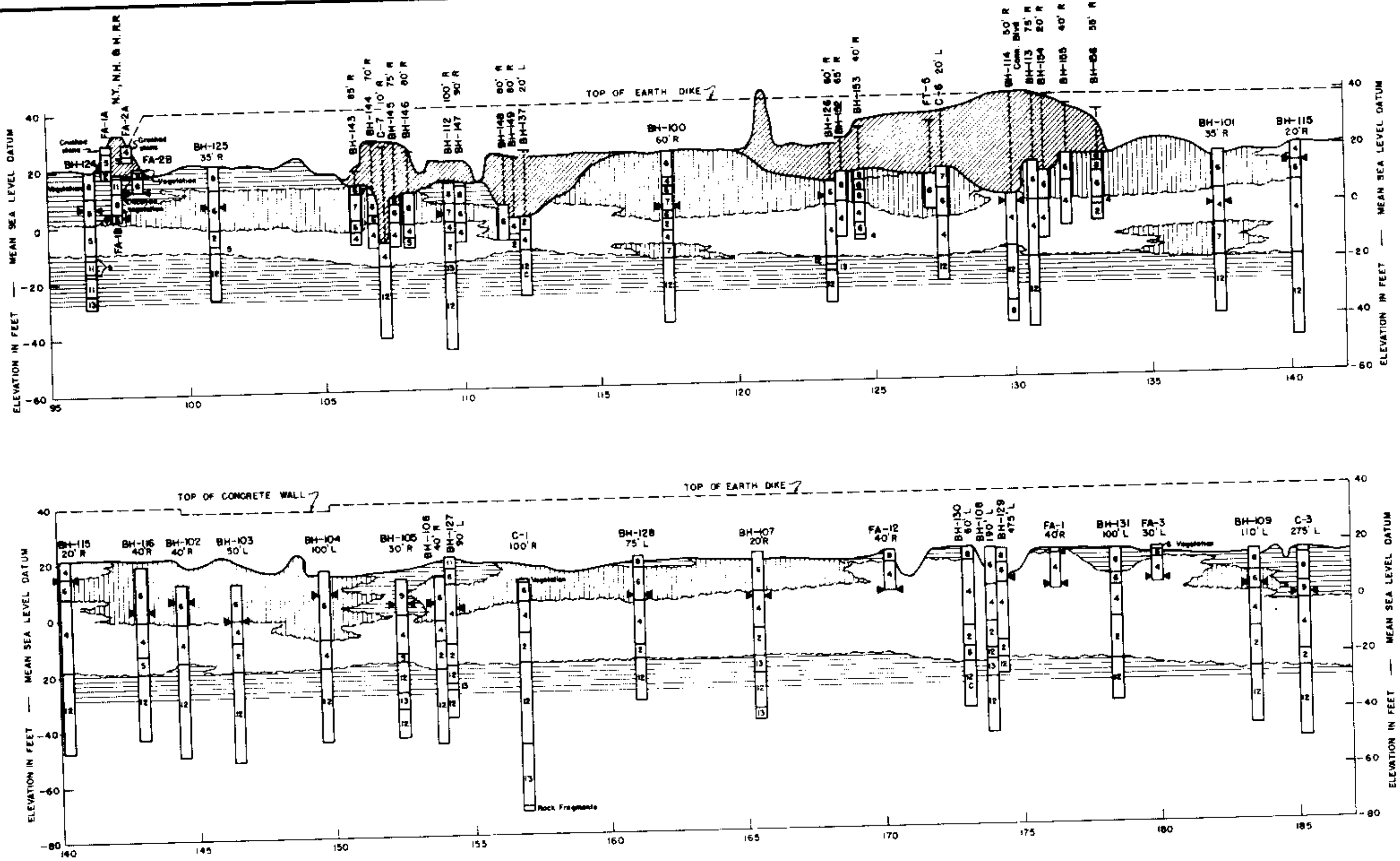
U. S. ENGINEER OFFICE, PROVIDENCE, R.I. MARCH 1939

SUBMITTED: GEOLOGIST, HEAD, GEOLOGY SECTION — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F.C. ENGINEERING DIV. — APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED — DRAWN BY — TRACED BY — CHECKED BY

FILE NO. CT-2-1131

PLATE NO. 7



LEGEND

BH – Drive sample bore hole by U S E D
C – Drive sample bore hole by contract
F T – Foundation test pit
F A – Foundation auger boring

- Artificial fill, consists of sand, gravel, and silt with bricks, cinders, and debris
- Pervious formation
- Moderately impervious formation
- Impervious formation
- Water table at time of exploration

NOTES:

Ground surface is that along ℄ of dike
Stationing shown is that of ℄ of dike.
L indicates landside of dike ℄
R indicates riverside of dike ℄.
For description of numerical classes see table

DATE	REVISION	REV.BY	CK.BY	AP.BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

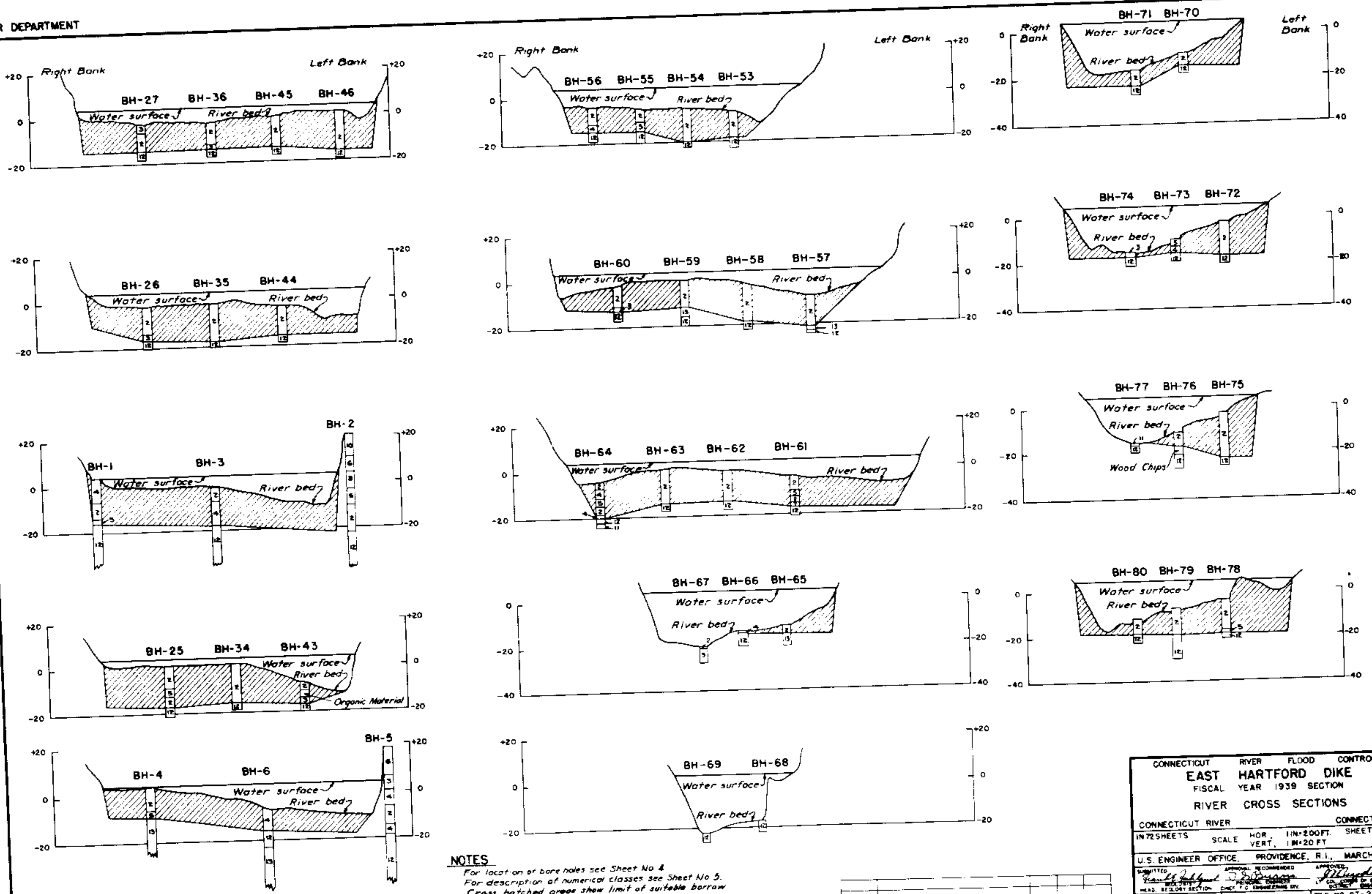
GEOLOGIC SECTION

CONNECTICUT RIVER — CONNECTICUT
IN SHEETS — HOR. SCALE: 1 IN. = 200 FT. VERT. SCALE: 1 IN. = 20 FT. — SHEET NO.
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939
SUBMITTED — APPROVAL RECOMMENDED — APPROVED:
HEAD, GEOLOGY SECTION — PRINCIPAL ENGINEER CHIEF, F.C. ENGINEERING DIV. — LT. COL., CORPS OF ENGINEERS DISTRICT ENGINEER
COMPILED — ASST. ENGINEER — DRAWN BY: W.A.S. JR. TRACED BY: G.B.C. CHECKED BY: P.H.
FILE NO. CT-2-1129

PLATE NO. 8

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



NOTES

For location of bore holes see Sheet No 4

For description of numerical classes see Sheet No 5.

Cross hatched areas show limit of suitable borrow

Elevations refer to Mean Sea Level Datum.

DATE	REVISION	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

RIVER CROSS SECTIONS

CONNECTICUT RIVER — CONNECTICUT

IN 72 SHEETS — SCALE HOR. 1 IN.=200 FT. VERT. 1 IN.=20 FT — SHEET NO. 8

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939

SUBMITTED: GEOLOGIST, HEAD, GEOLOGY SECTION — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF F.C. ENGINEERING DIV. — APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: — DRAWN BY: F.H.S. TRACED BY: W.M.M. CHECKED BY: T.W.J. — FILE NO. CT-2-1117

8

PLATE NO. 9

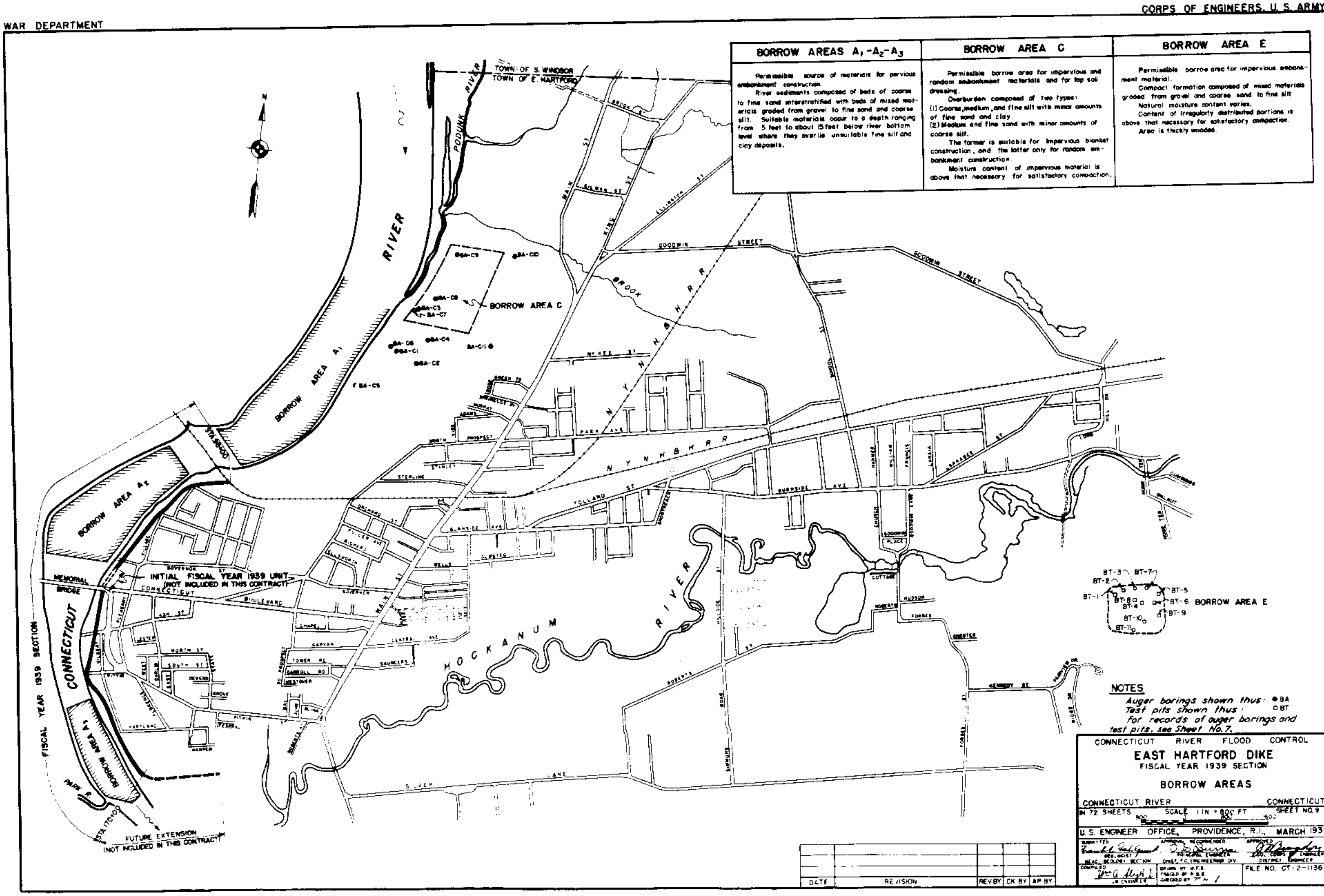
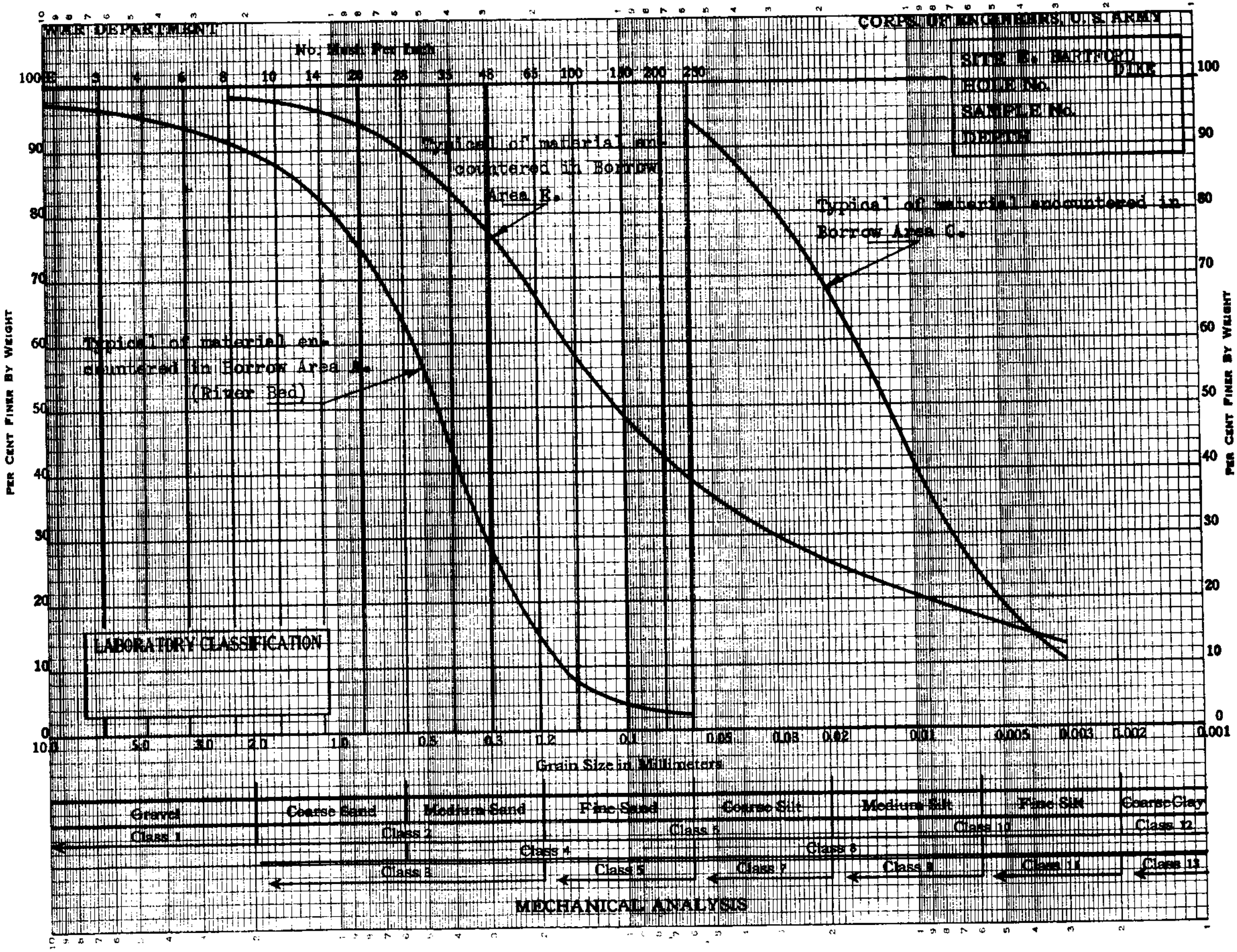

WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY
BORROW AREAS A_1-A_2-A_3
Permissible source of materials for pervious embankment construction.
River sediments composed of beds of coarse to fine sand interstratified with beds of mixed materials graded from gravel to fine sand and coarse silt. Suitable materials occur to a depth ranging from 5 feet to about 15 feet below river bottom level where they overlie unsuitable fine silt and clay deposits.
BORROW AREA C
Permissible borrow area for impervious and random embankment materials and for top soil dressing.
Overburden composed of two types:
(1) Coarse, medium, and fine silt with minor amounts of fine sand and clay.
(2) Medium and fine sand with minor amounts of coarse silt.
The former is suitable for impervious blanket construction, and the latter only for random embankment construction.
Moisture content of impervious material is above that necessary for satisfactory compaction.
BORROW AREA E
Permissible borrow area for impervious embankment material.
Compact formation composed of mixed materials graded from gravel and coarse sand to fine silt.
Natural moisture content varies.
Content of irregularly distributed portions is above that necessary for satisfactory compaction.
Area is thickly wooded.
TOWN OF S. WINDSOR
TOWN OF E. HARTFORD
PODUNK RIVER
CONNECTICUT RIVER
BORROW AREA A1
BORROW AREA A2
BORROW AREA A3
BORROW AREA C
BORROW AREA E
HOCKANUM RIVER
MEMORIAL BRIDGE
INITIAL FISCAL YEAR 1939 UNIT
(NOT INCLUDED IN THIS CONTRACT)
FISCAL YEAR 1939 SECTION
FUTURE EXTENSION
(NOT INCLUDED IN THIS CONTRACT)
GOODWIN STREET
SILVER LANE
N Y N H & H R R
NOTES
Auger borings shown thus: ● BA
Test pits shown thus: ▫ BT
For records of auger borings and test pits, see Sheet No. 7.
CONNECTICUT RIVER FLOOD CONTROL
EAST HARTFORD DIKE
FISCAL YEAR 1939 SECTION
BORROW AREAS
CONNECTICUT RIVER
CONNECTICUT
IN 72 SHEETS
SCALE 1 IN = 800 FT
SHEET NO. 9
U. S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939
FILE NO. CT-2-1136
DATE
REVISION
REV BY
CK BY
AP BY

PLATE NO. 10



WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
SITE E. HARTFORD DIKE
HOLE No.
SAMPLE No.
DEPTH
No. Mesh Per Inch
Typical of material encountered in Borrow Area E.
Typical of material encountered in Borrow Area C.
Typical of material encountered in Borrow Area
(River Bed)
LABORATORY CLASSIFICATION
Per Cent Finer By Weight
Grain Size in Millimeters
Gravel
Coarse Sand
Medium Sand
Fine Sand
Coarse Silt
Medium Silt
Fine Silt
Coarse Clay
Class 1
Class 2
Class 3
Class 4
Class 5
Class 6
Class 7
Class 8
Class 9
Class 10
Class 11
Class 12
Class 13
MECHANICAL ANALYSIS

PLATE NO. 11.

PLATE NO. 12

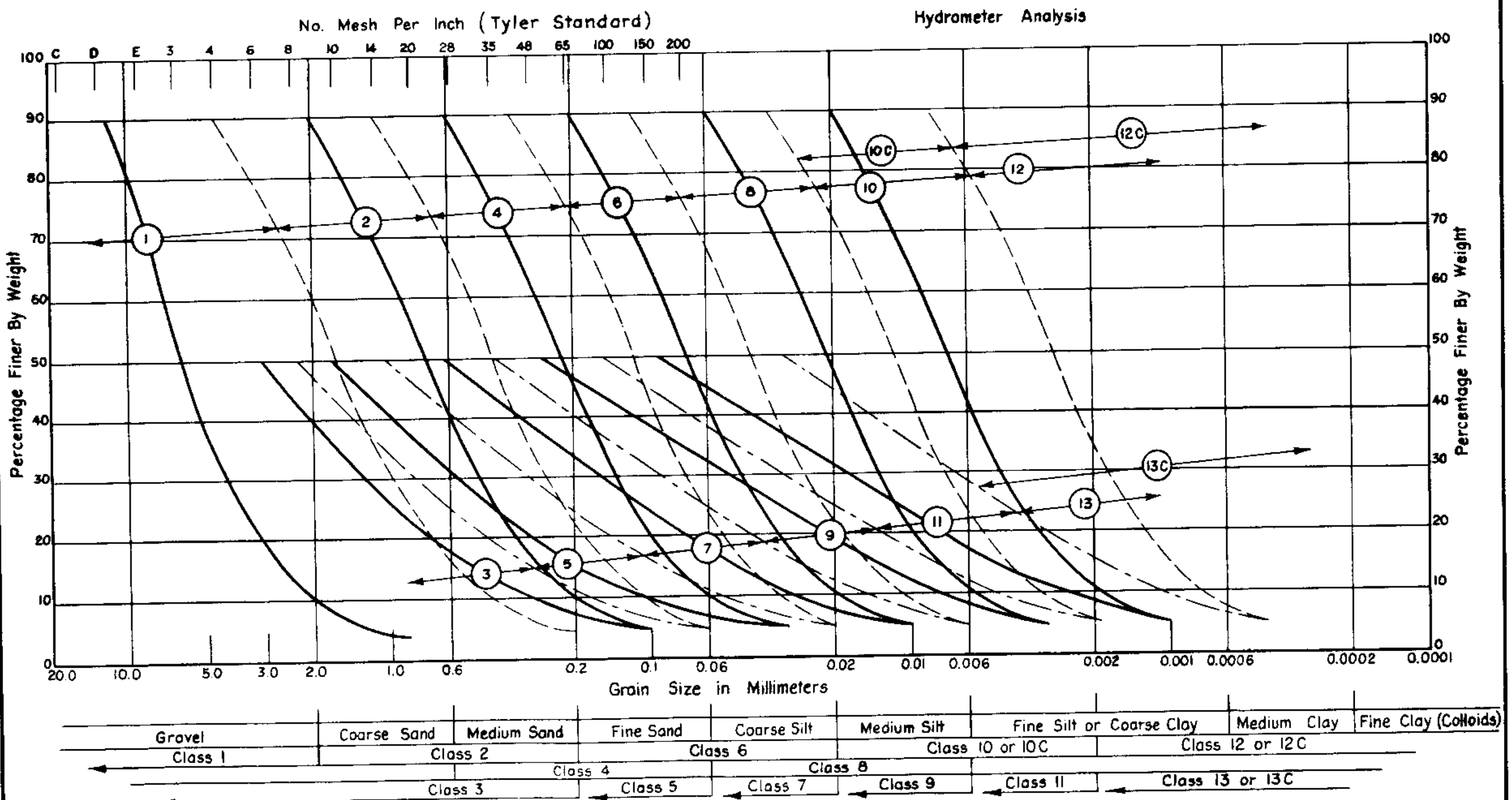


DIAGRAM SHOWING LIMITS OF SOIL CLASSES

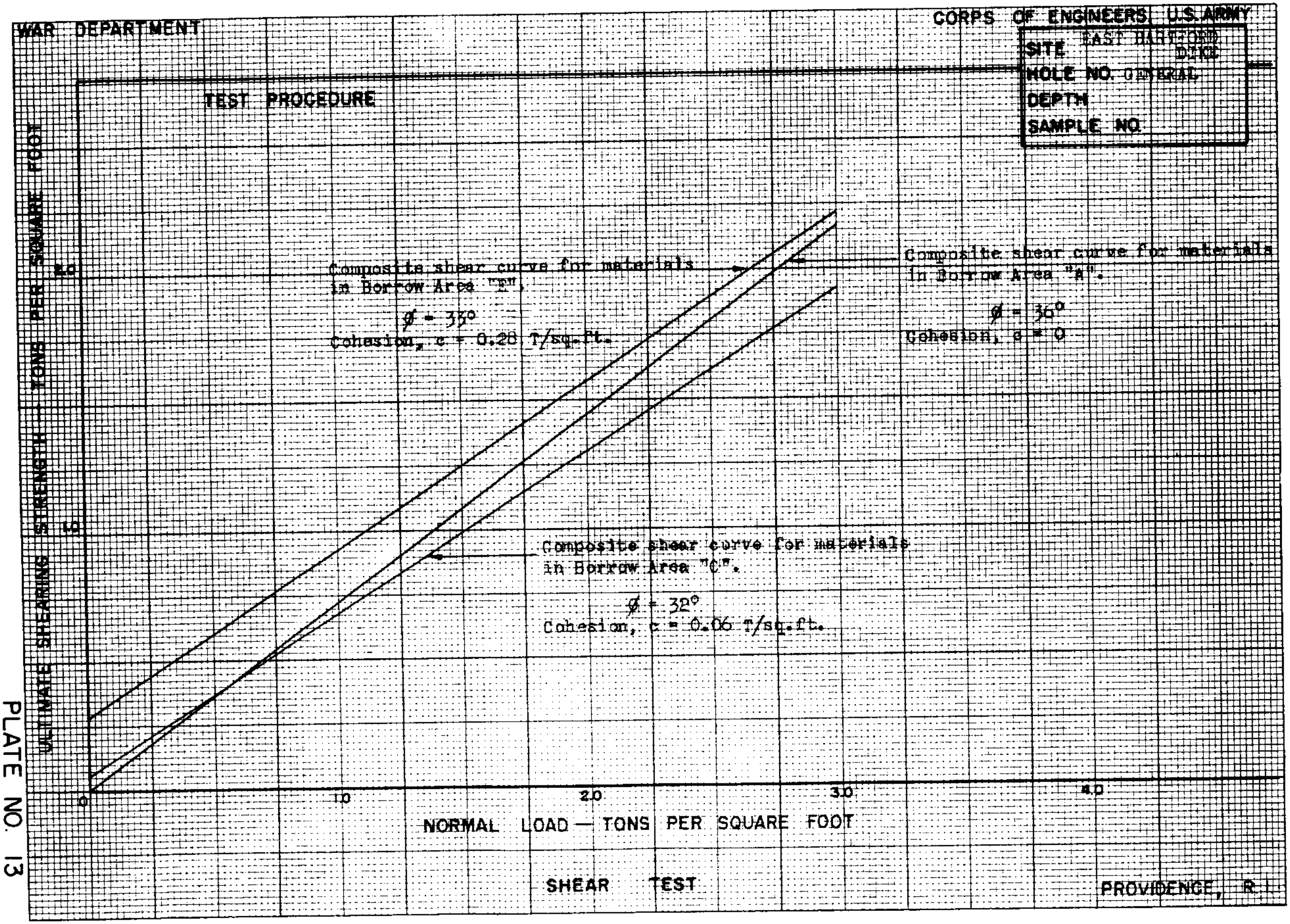

WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
SITE EAST HARTFORD DIKE
HOLE NO. GENERAL
DEPTH
SAMPLE NO.
TEST PROCEDURE
Composite shear curve for materials in Borrow Area "F".
ø = 33°
Cohesion, c = 0.28 T/sq.ft.
Composite shear curve for materials in Borrow Area "A".
ø = 36°
Cohesion, c = 0
Composite shear curve for materials in Borrow Area "C".
ø = 32°
Cohesion, c = 0.06 T/sq.ft.
ULTIMATE SHEARING STRENGTH — TONS PER SQUARE FOOT
2.0
1.0
0
1.0
2.0
3.0
4.0
NORMAL LOAD — TONS PER SQUARE FOOT
SHEAR TEST
PROVIDENCE, R.I.

PLATE NO. 13

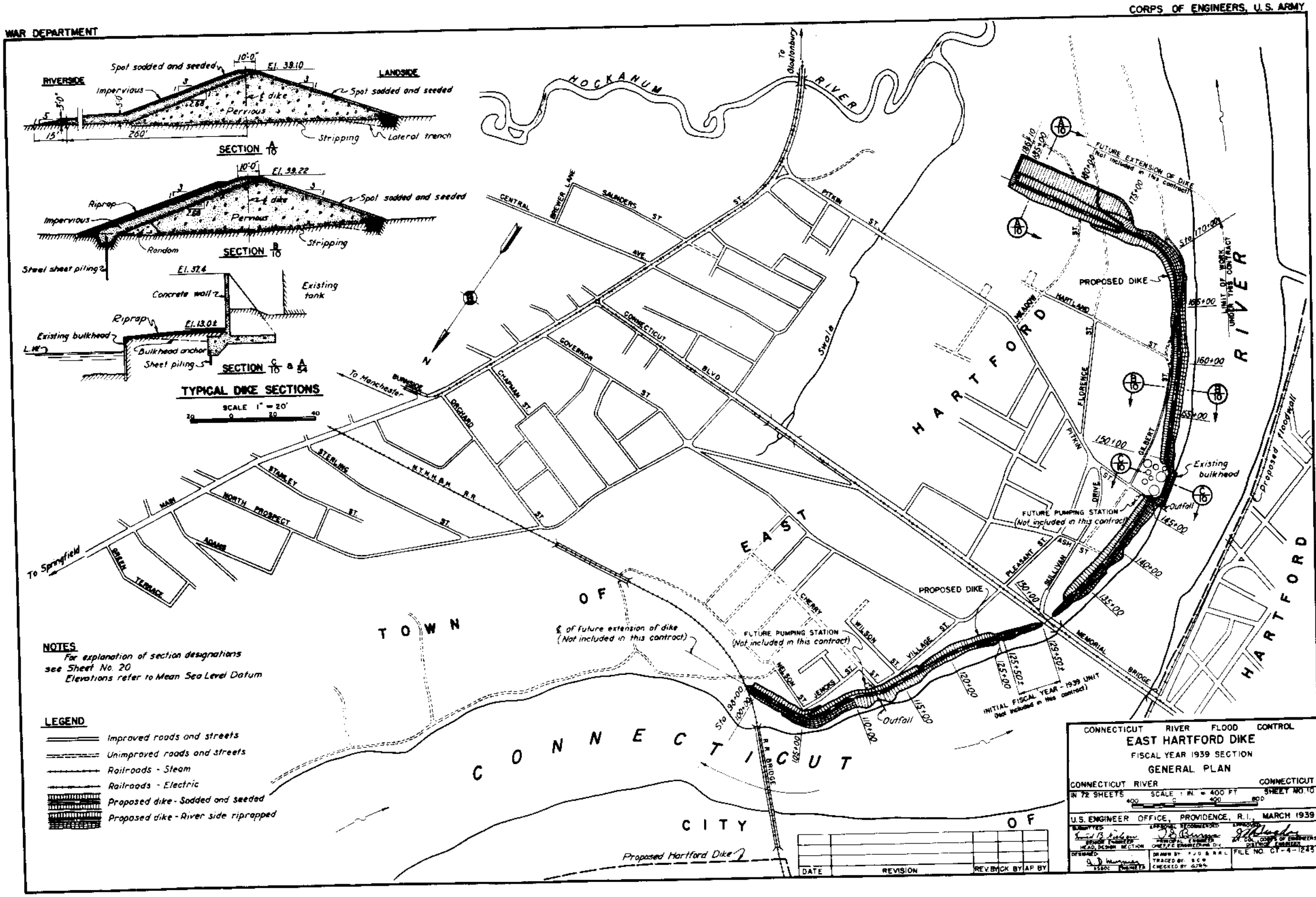


PLATE NO. 14

(10)

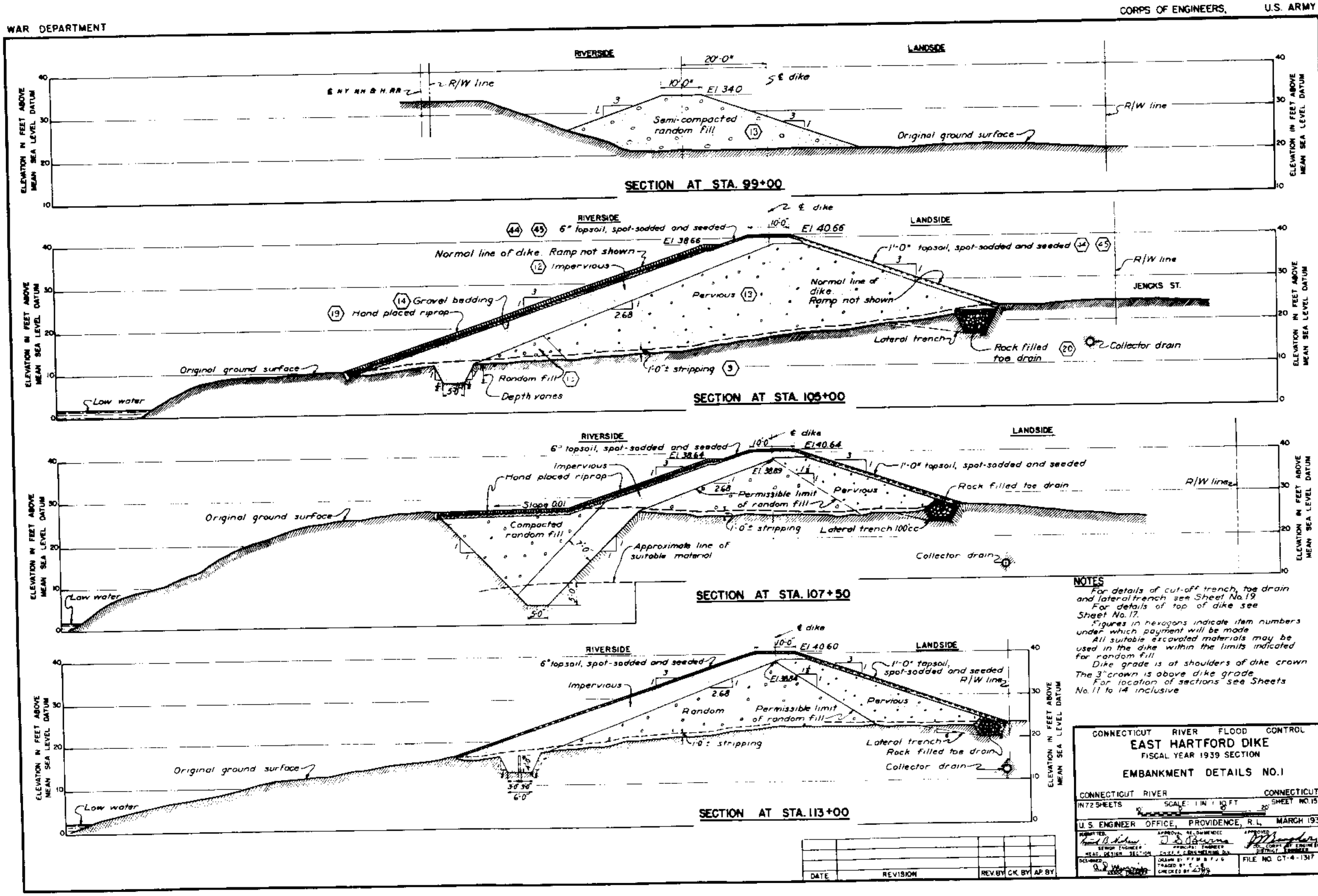


PLATE NO. 15

(15)

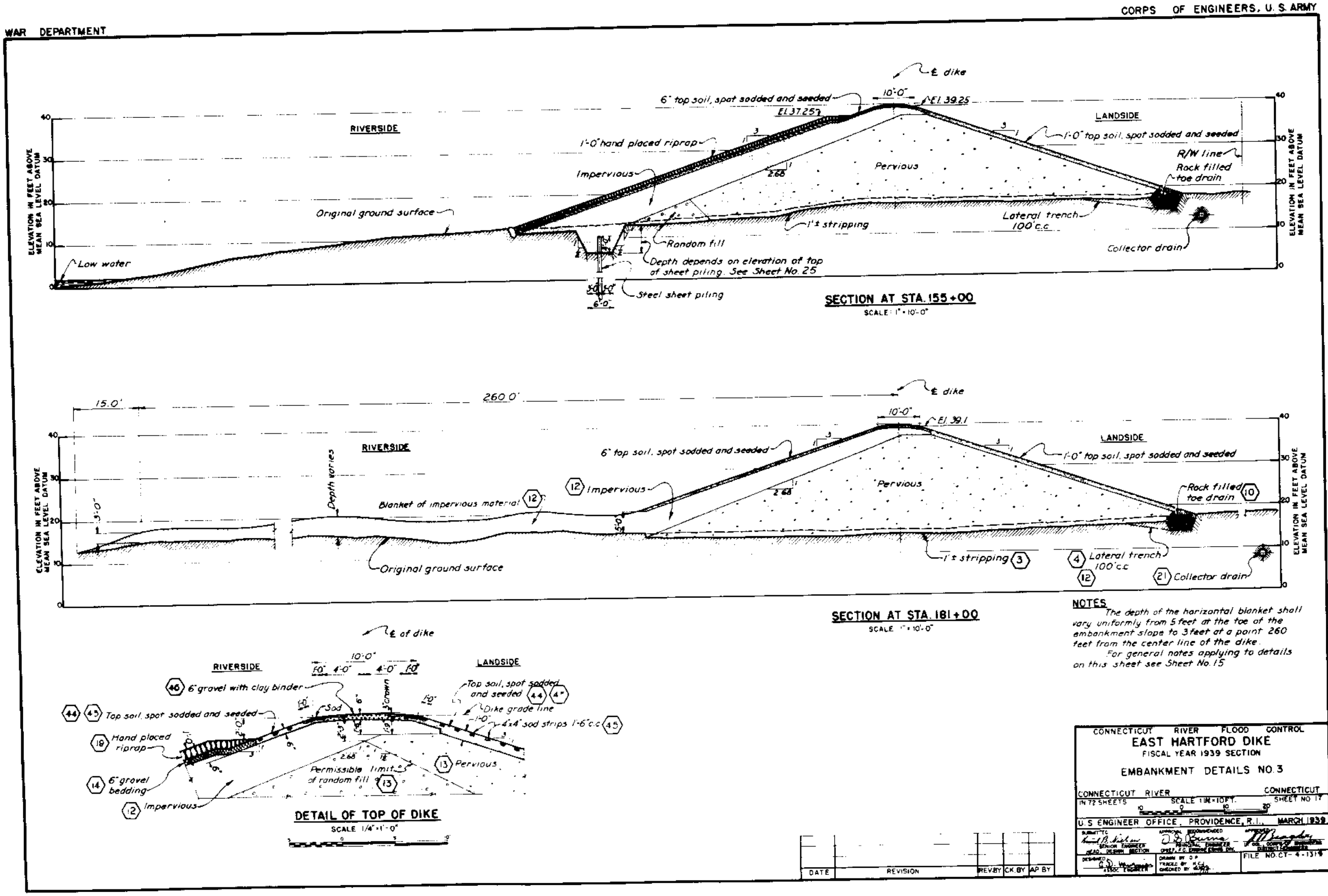


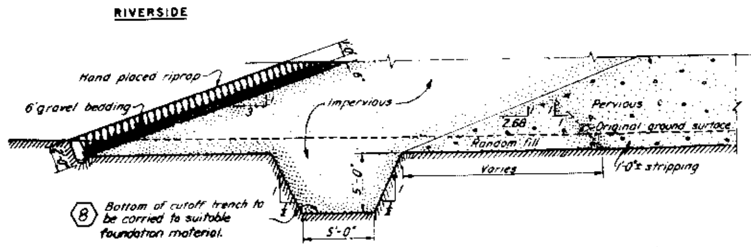
PLATE NO. 16

(17)

WAR DEPARTMENT

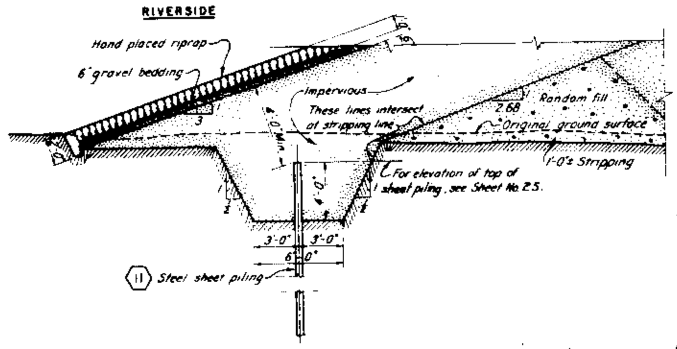
CORPS OF ENGINEERS, U.S. ARMY

PLATE NO 17



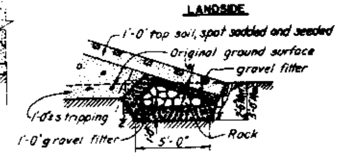
TYPICAL CUTOFF TRENCH

Without sheet piling

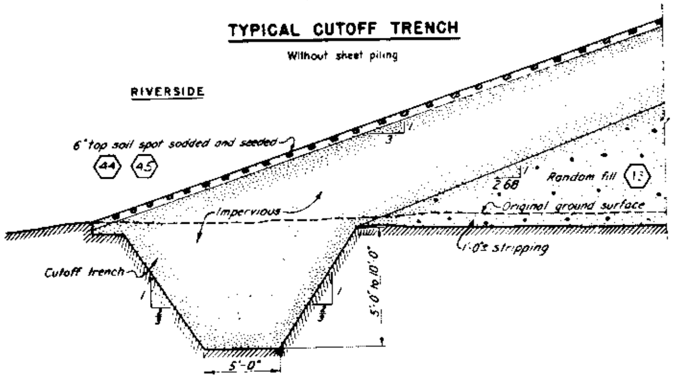


CUTOFF TRENCH

With sheet piling

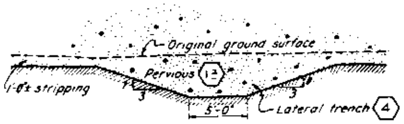


2'-6" TO 5'-0" DEPTHS



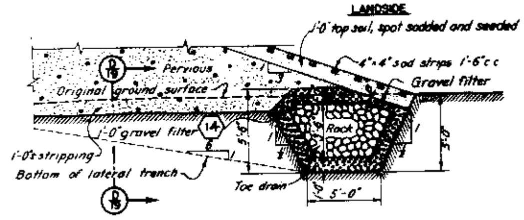
SPECIAL CUTOFF TRENCH

Depth: 5ft to 10ft - Without sheet piling

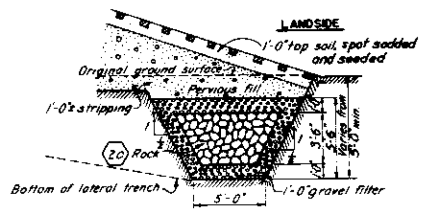


SECTION D/19

Trenches to be spaced at 100' c c longitudinally

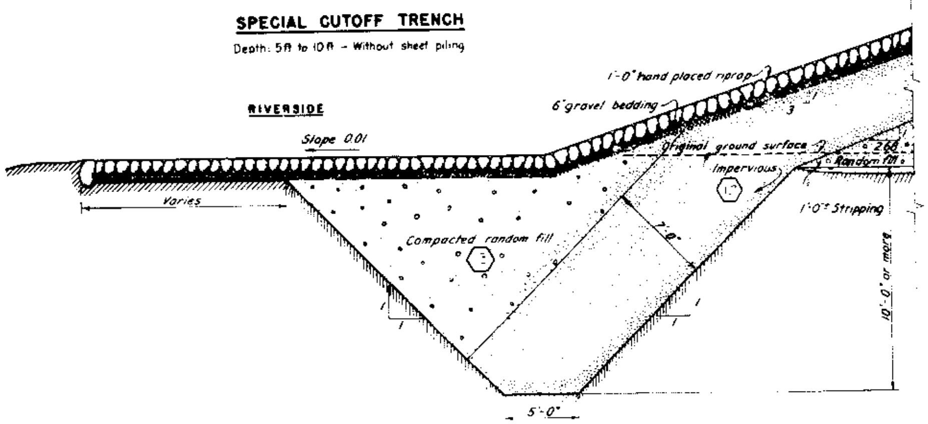


5'-0" DEPTH



5'-0" AND GREATER DEPTHS

TYPICAL TOE DRAIN CROSS SECTIONS



SPECIAL CUTOFF TRENCH

Depth Over 10 ft - Without sheet piling

NOTES

The depth of cutoff trench will be determined by the Contracting Officer.

For general notes applying to details on this sheet see Sheet No 15.

DATE	REVISION	REV.BY	CK.BY	AP.BY

CONNECTICUT RIVER FLOOD CONTROL
EAST HARTFORD DIKE
FISCAL YEAR 1939 SECTION

EMBANKMENT DETAILS NO.5

CONNECTICUT RIVER — CONNECTICUT

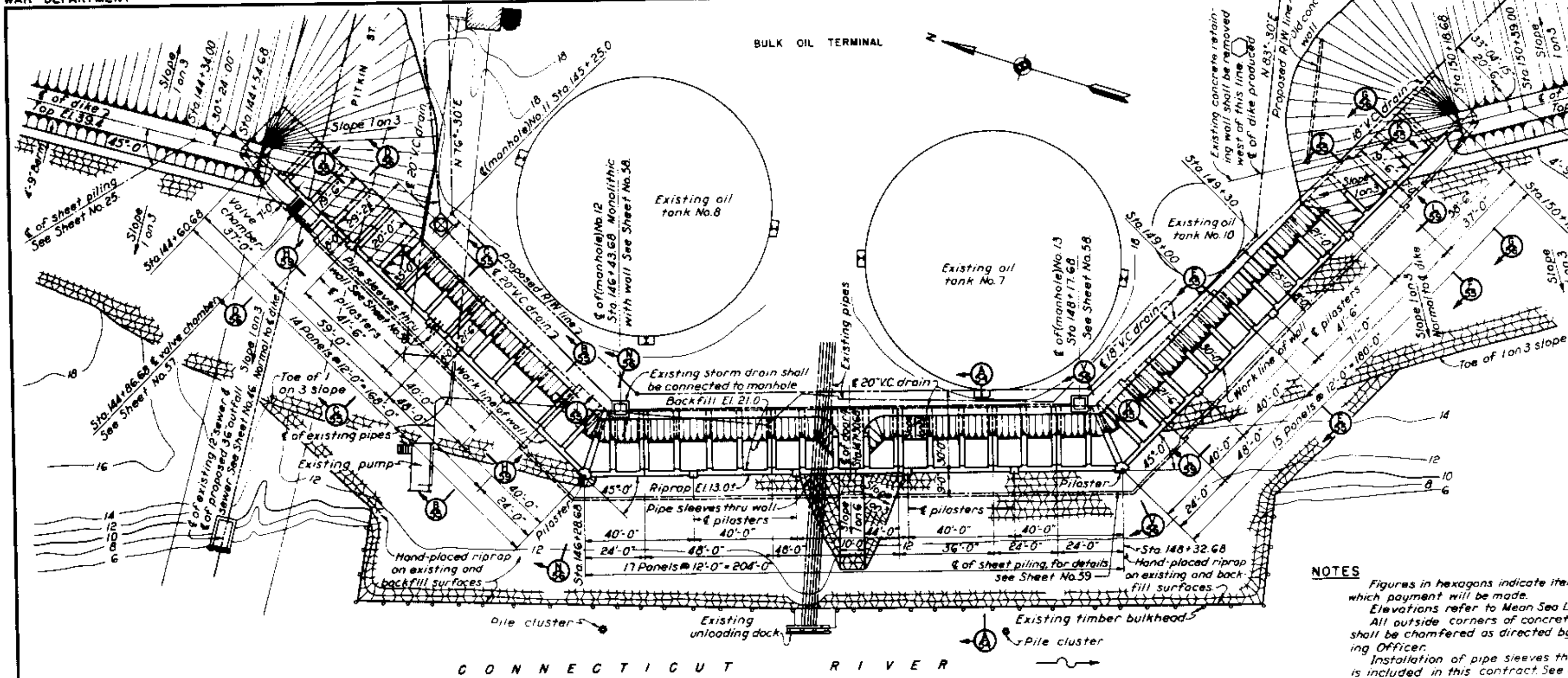
IN 72 SHEETS — SCALE: 1/4 IN. = 1 FT. — SHEET NO. 19

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939

FILE NO. CT-4-1321

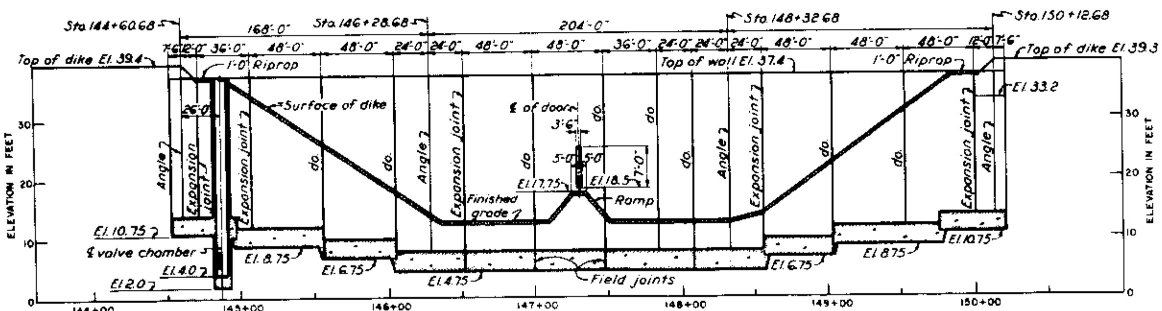
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



PLAN

SCALE 1" = 20'



DEVELOPED PROFILE ALONG FACE OF WALL

SCALE HOR. 1" = 40' VERT. 1" = 10'

NOTES

Figures in hexagons indicate item number under which payment will be made.

Elevations refer to Mean Sea Level Datum.

All outside corners of concrete surfaces shall be chamfered as directed by the Contracting Officer.

Installation of pipe sleeves through the wall is included in this contract. See detail Sheet No. 58.

All concrete shown on this sheet shall be Class "A" and payment will be made under Item No. 59

For details of expansion joints see Sheet No. 54.

For details of architectural treatment of wall see Sheet No 59.

For details of door and accessories see Sheet No. 60.

For details of valve chamber for outfall see Sheet No. 57.

For layouts of drainage systems see Sheets No. 26 to No. 28 incl.

For details of sheet pile cutoff see Sheets No. 18, 25, 57 and 59.

DATE	REVISION	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

FISCAL YEAR 1939 SECTION

CONCRETE FLOODWALL

PLAN AND PROFILE

CONNECTICUT RIVER — CONNECTICUT

IN 72 SHEETS — SCALE 1 IN. = 20 FT. — SHEET NO 53

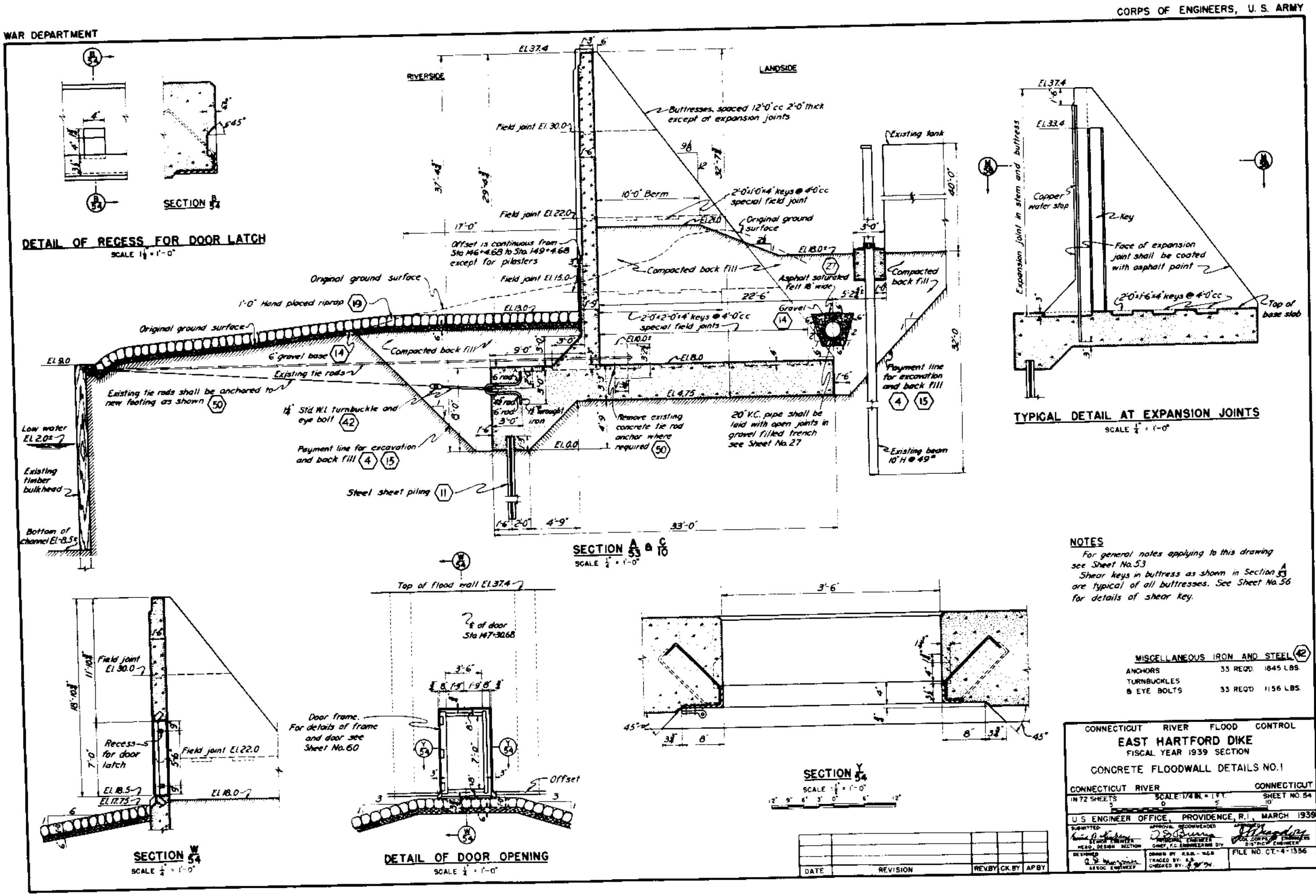
U. S. ENGINEER OFFICE, PROVIDENCE, R. I., MARCH 1939

SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION — APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, F. C. ENGINEERING DIV. — APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: ASSOC. ENGINEER — DRAWN BY R.T.M.-W.C.B. TRACED BY J.T. CHECKED BY — FILE NO CT-4-1355

53

PLATE NO. 18

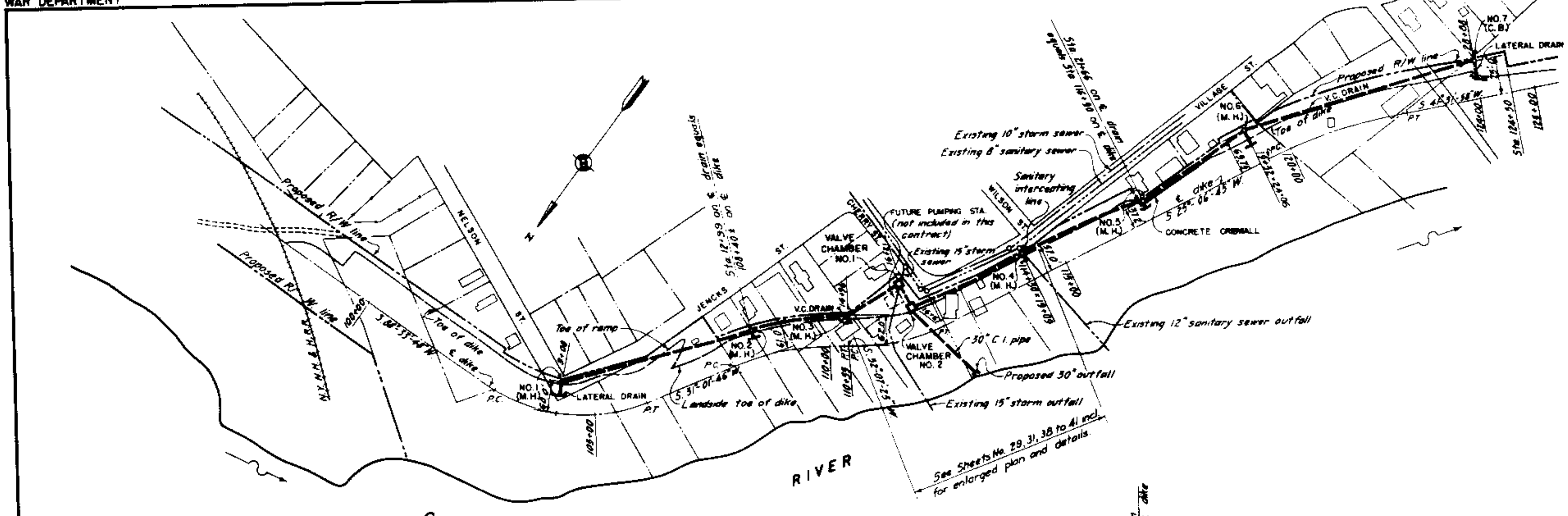


WAR DEPARTMENT
CORPS OF ENGINEERS, U. S. ARMY
DETAIL OF RECESS FOR DOOR LATCH
SCALE 1½" = 1'-0"
SECTION B/54
RIVERSIDE
LANDSIDE
EL 37.4
Buttresses spaced 12'-0" cc 2'-0" thick except at expansion joints
Field joint El. 30.0
Field joint El. 22.0
Field joint El. 15.0
10'-0" Berm
2'-0"x1'-0"x4" keys @ 4'-0" cc special field joint
Original ground surface
Offset is continuous from Sta. 146+4.68 to Sta. 149+4.68 except for pilasters
Compacted back fill
Asphalt saturated felt 18" wide
Existing tank
1'-0" Hand placed riprap
Original ground surface
6" gravel base
Gravel
2'-0"x2'-0"x4" keys @ 4'-0" cc special field joints
Existing tie rods shall be anchored to new footing as shown
Existing tie rods
1¼" Std. W.I. turnbuckle and eye bolt
Payment line for excavation and back fill
Remove existing concrete tie rod anchor where required
20" V.C. pipe shall be laid with open joints in gravel filled trench see Sheet No. 27
Payment line for excavation and back fill
Existing beam 10" H @ 49#
Low water EL 2.0±
Existing timber bulkhead
Bottom of channel El. -8.5±
Steel sheet piling
SECTION A/53 & C/10
SCALE ¼" = 1'-0"
Copper water stop
Key
Face of expansion joint shall be coated with asphalt paint
2'-0"x1'-6"x4" keys @ 4'-0" cc
Top of base slab
Expansion joint in stem and buttress
TYPICAL DETAIL AT EXPANSION JOINTS
SCALE ¼" = 1'-0"
NOTES
For general notes applying to this drawing see Sheet No. 53
Shear keys in buttress as shown in Section A/53 are typical of all buttresses. See Sheet No. 56 for details of shear key.
MISCELLANEOUS IRON AND STEEL (42)
ANCHORS 33 REQD. 1845 LBS.
TURNBUCKLES & EYE BOLTS 33 REQD. 1156 LBS.
Field joint El. 30.0
Recess for door latch
Field joint El. 22.0
El. 18.5
El. 17.75
El. 18.0
SECTION W/54
SCALE ¼" = 1'-0"
Top of flood wall El. 37.4
℄ of door Sta. 147+30.68
Door frame. For details of frame and door see Sheet No. 60
Offset
DETAIL OF DOOR OPENING
SCALE ¼" = 1'-0"
SECTION Y/54
SCALE 1½" = 1'-0"
DATE
REVISION
REV. BY
CK. BY
AP. BY
CONNECTICUT RIVER FLOOD CONTROL
EAST HARTFORD DIKE
FISCAL YEAR 1939 SECTION
CONCRETE FLOODWALL DETAILS NO. 1
CONNECTICUT RIVER
CONNECTICUT
IN 72 SHEETS
SCALE: 1/4 IN. = 1 FT.
SHEET NO. 54
U. S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939
FILE NO. CT.-4-1356
54

PLATE NO. 19

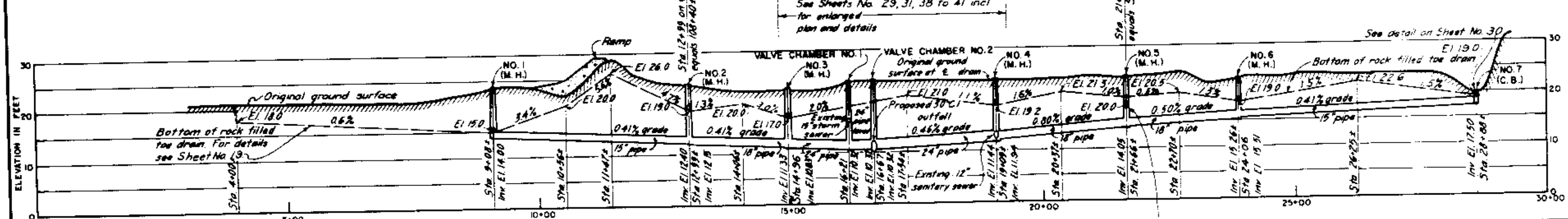
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



PLAN

SCALE: 1" = 100'



PROFILE ON ℄ COLLECTOR DRAIN

SCALE: HOR., 1" = 100'
VERT., 1" = 10'

Lateral drain to be dropped 2'-0" to clear cr b wall see Sheet No 24

NOTES

All pipes shall be vitrified clay unless otherwise noted.

Elevations refer to Mean Sea Level Datum.

The drainage systems will be paid for at the contract price under the following respective items Item No 2, 9, 16, 22 to 34 incl., 36, 39, 40 and 42.

See Sheet No 29 for manhole and catch-basin schedule and payment.

Offset distances to drains are from ℄ of dike to center of manhole. Distances are approximate and measured normal to ℄ of dike or radially on curves.

DATE	REVISION	REV BY	CK BY	AP BY

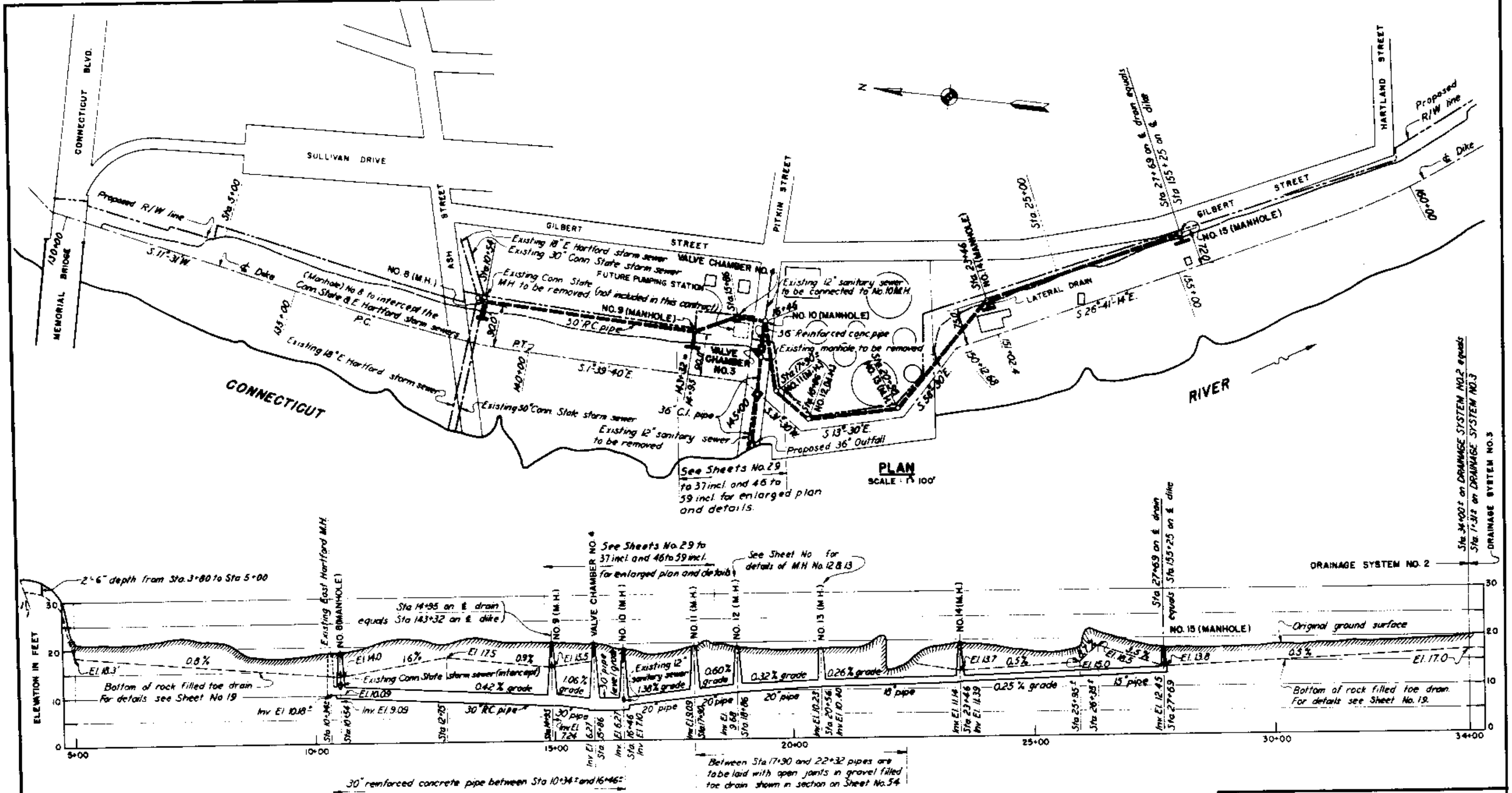
CONNECTICUT RIVER FLOOD CONTROL
EAST HARTFORD DIKE
FISCAL YEAR 1939 SECTION
DRAINAGE SYSTEM NO. 1

CONNECTICUT RIVER — CONNECTICUT
IN 72 SHEETS — SCALE 1IN = 100 FT. — SHEET NO. 26
U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939

SUBMITTED: SENIOR ENGINEER, HEAD, DESIGN SECTION
APPROVAL RECOMMENDED: PRINCIPAL ENGINEER, CHIEF, E.C. ENGINEERING DIV.
APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER
DESIGNED: ASSOC. ENGINEER
DRAWN BY: F. A. B. TRACED BY: W. E. L. CHECKED BY: C.A.W.
FILE NO. CT-4-1328

(26)

PLATE NO. 20



PROFILE ON ℄ COLLECTOR DRAIN

SCALE: HOR 1"=100' VERT. 1"=10'

NOTES

For notes applying to details on this sheet see Sheet No. 26

Elevations refer to Mean Sea Level Datum

DATE	REVISION	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL
EAST HARTFORD DIKE
FISCAL YEAR 1939 SECTION
DRAINAGE SYSTEM NO. 2

CONNECTICUT RIVER, CONNECTICUT
IN 72 SHEETS — SCALE 1 IN.=100 FT. — SHEET NO. 27

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MARCH 1939

SUBMITTED — APPROVAL RECOMMENDED — APPROVED
SENIOR ENGINEER HEAD, DESIGN SECTION — PRINCIPAL ENGINEER CHIEF, F.C. ENGINEERING DIV. — LT. COL., CORPS OF ENGINEERS DISTRICT ENGINEER
DESIGNED — ASSOC. ENGINEER
DRAWN BY — TRACED BY — CHECKED BY
FILE NO. CT-4-1329

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

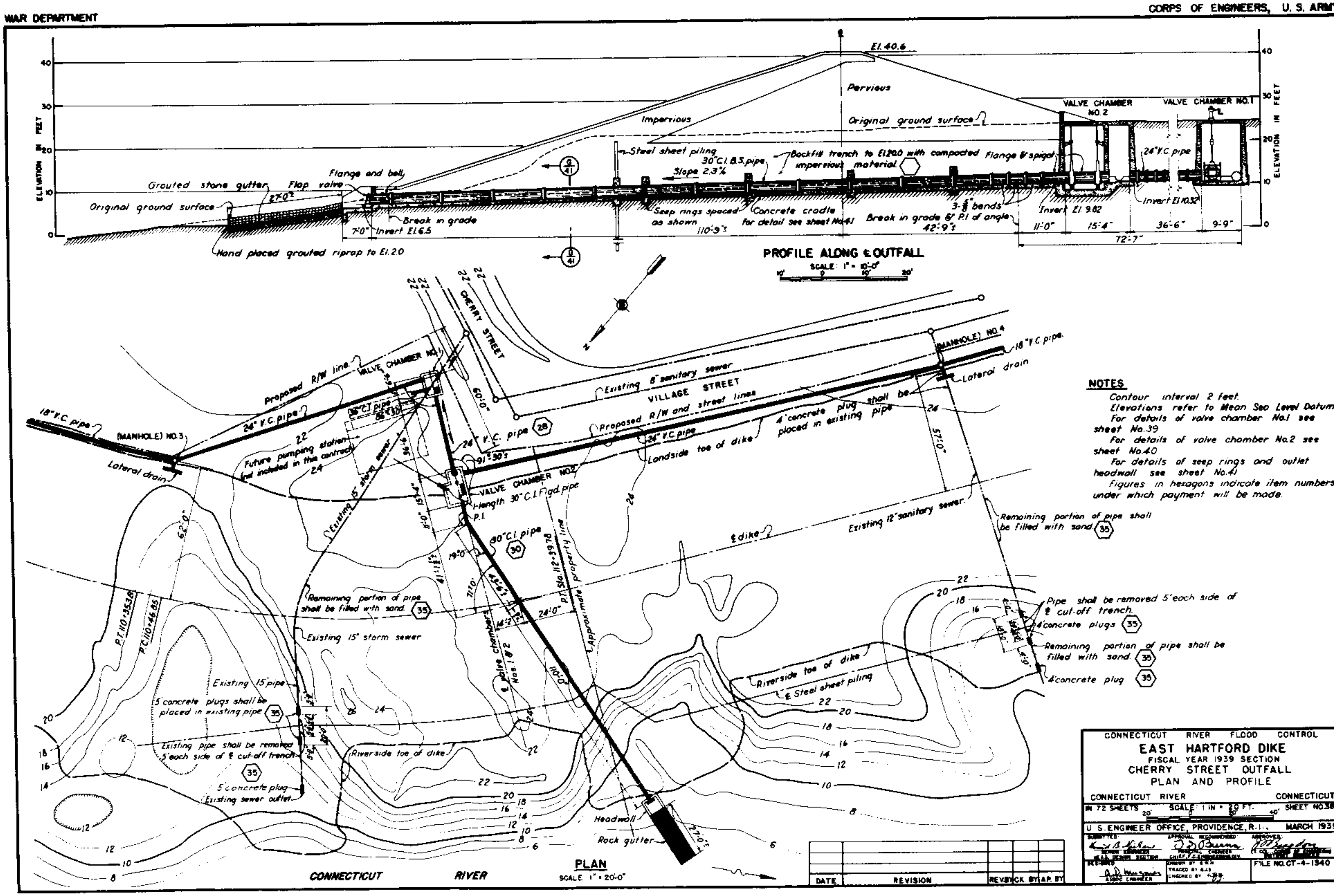
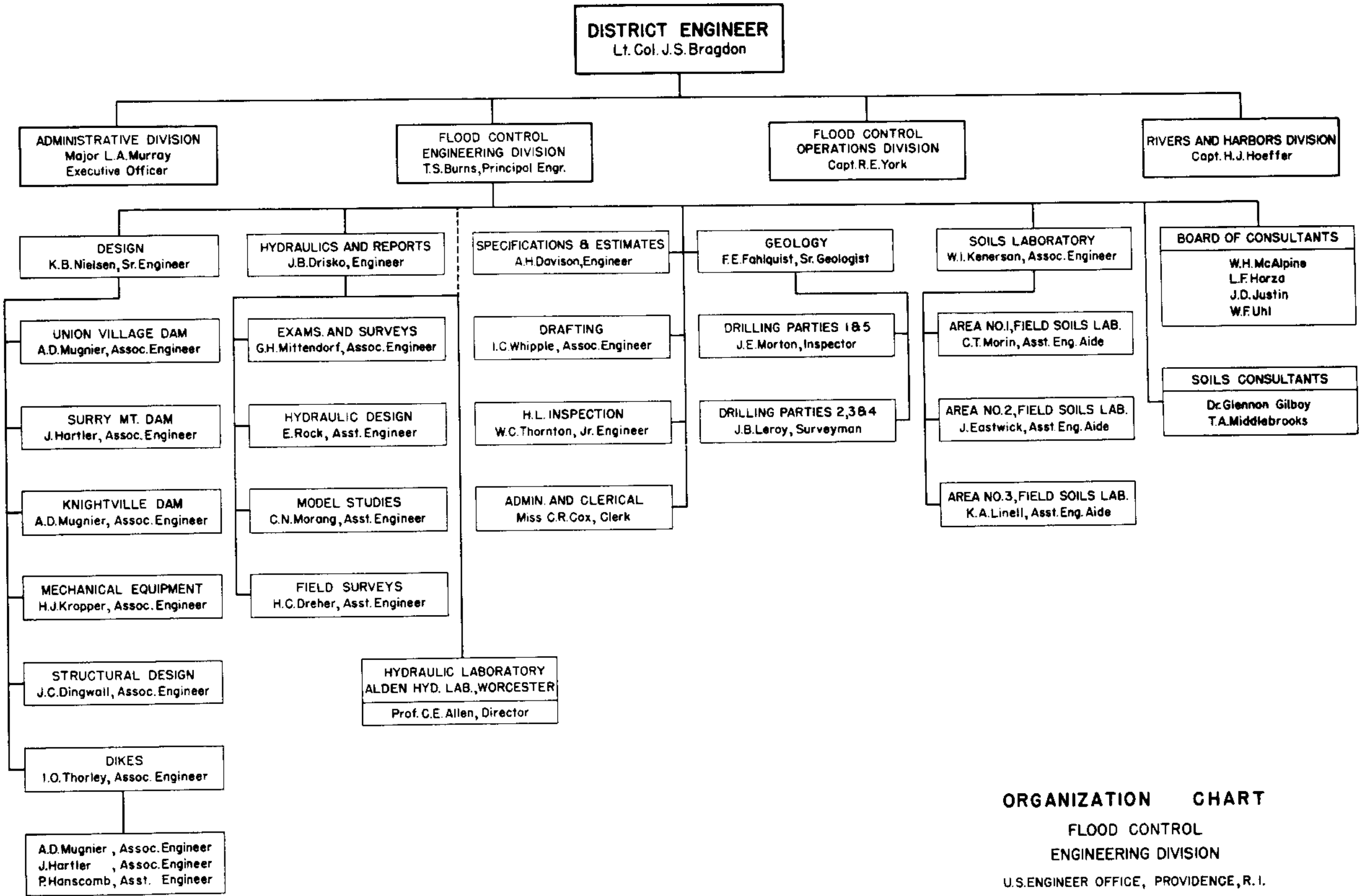


PLATE NO. 22

(38)

PLATE NO. 23



ORGANIZATION CHART

FLOOD CONTROL

ENGINEERING DIVISION

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

FEBRUARY, 1939

CONNECTICUT RIVER FLOOD CONTROL

EAST HARTFORD DIKE

CONNECTICUT RIVER CONNECTICUT

FISCAL YEAR 1939 SECTION – CONTRACT

(R.R. SOUTH ALONG CONNECTICUT RIVER)

EH. 2

ANALYSIS OF DESIGN

1939

APPENDIX A

CORPS OF ENGINEERS, U.S. ARMY

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

APPENDIX A - CONCRETE AND STEEL COMPUTATIONS

WAR DEPARTMENT

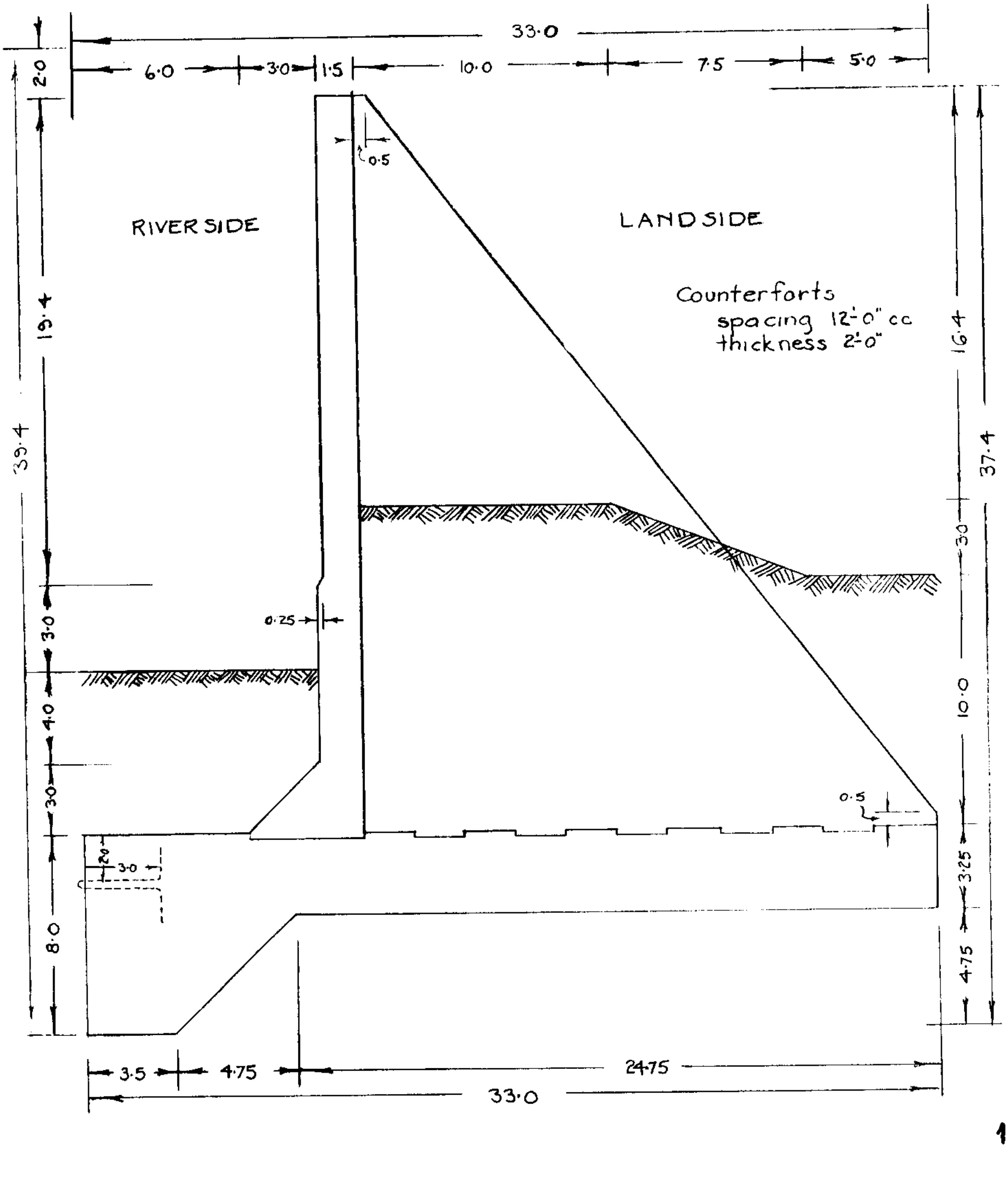
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford— Shell Oil Co- Section "A"

Computation Sectional Dimensions

Computed by CWEM **Checked by** SHK **Date** 3-10-39



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page . . . . . .

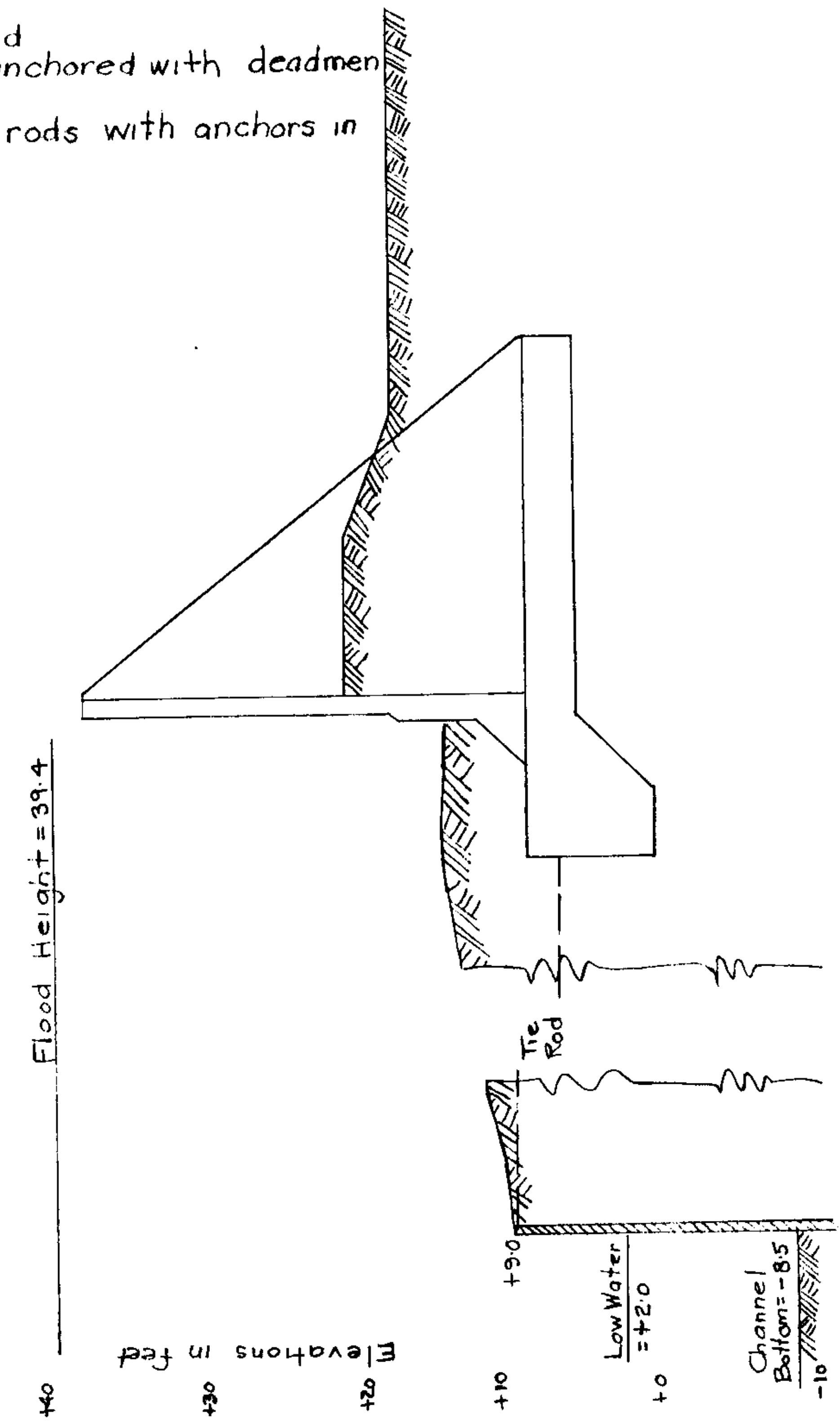
Subject East Hartford - Shell Oil Co - Section "A"

Computation Relation of Bulkhead to Wall

Computed by Curtem **Checked by** L.A.R. **Date** 3-7-3?

Timber bulkhead
at present anchored with deadmen

use same tie rods with anchors in
concrete



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. **Page** ....

Subject East Hartford-Shell Oil Co - Section "A"

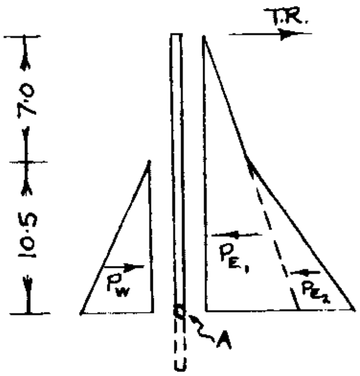
Computation Tie rod Pull

Computed by C.W.R.M. **Checked by** J.S.M. **Date** 3-7-29

Since tie rod pull is a direct factor in the stability of the wall, let us compute its value first.

Assume the hinge point of the bulkhead at the channel bottom.

Case II River down

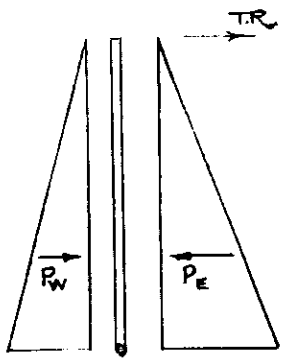


Moments

		↷	↶
P_W	$\frac{1}{6} \times 62.5' \times 10.5'^3$	= 72,350'	
P_{E_1}	$\frac{1}{6} \times 35' \times 17.5^{3'}$	:	187,580'
P_{E_2}	$\frac{1}{6} \times (80-35) \times 10.5'^3$	:	52,090'
Total Moment =	$\frac{1}{6} \times$		167,320' = 27,886'#↶

Tie rod pull $= \frac{27,886}{17.5} =$ 1593'#/running foot of wall.

Case I River at flood.

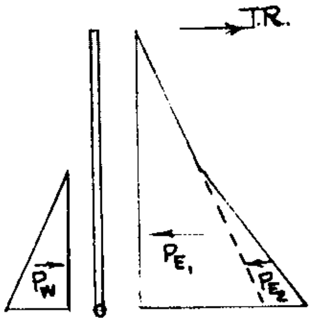


Moments

		↷	↶
P_W =	$\frac{1}{6} \times 62.5' \times 17.5^3$ =	334,960'	
P_E =	$\frac{1}{6} \times 80.0' \times 17.5^{3'}$ =		428,750'
Total Moment =	$\frac{1}{6} \times$		93,790' = 15,632'#↶

Tie rod pull $= \frac{15,632}{17.5} =$ 895'#/running foot of wall

Case III - River receded suddenly



Moments

		↷	↶
P_W	$\frac{1}{6} \times 62.5' \times 10.5'^3$	= 72,350'	
P_{E_1}	$\frac{1}{6} \times 50' \times 17.5^{3'}$	=	267,970'
P_{E_2}	$\frac{1}{6} \times (80-50) \times 10.5^{3'}$	=	34,730'
Total Moment =	$\frac{1}{6} \times$		230,350' = 38,392'#↶

Tie rod pull $= \frac{38,392}{17.5} =$ 2,194'#/running ft of wall

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Co - Section "A"

Computation Bulkhead tierod details

Computed by CWB **Checked by** AHB. **Date** 3-15-39

Since the spacing of the tierods in place is not known at the time of designing, the maximum allowable spacing of tie rods, using a conventional hook, will be shown.

Detail calls for $1\frac{1}{2}''$ wrought iron tie rod.

Working stress, W.I. = 16,000 #/□ ; Net area = 1.30 □"

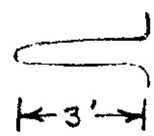
Force per rod = 16,000 × 1.30 = 20,800 #

Maximum pull per running foot (pp 3) = 2194 #/ft

Maximum allowable spacing = $\frac{20{,}800}{2194}$ = 9.5 = approx 10 ft

---

Check on bond --- 36" embedment - double rod



$$U = \frac{20{,}800^{\#}}{2 \times 36'' \times 4.7''} = 64^{\#}/\square''$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

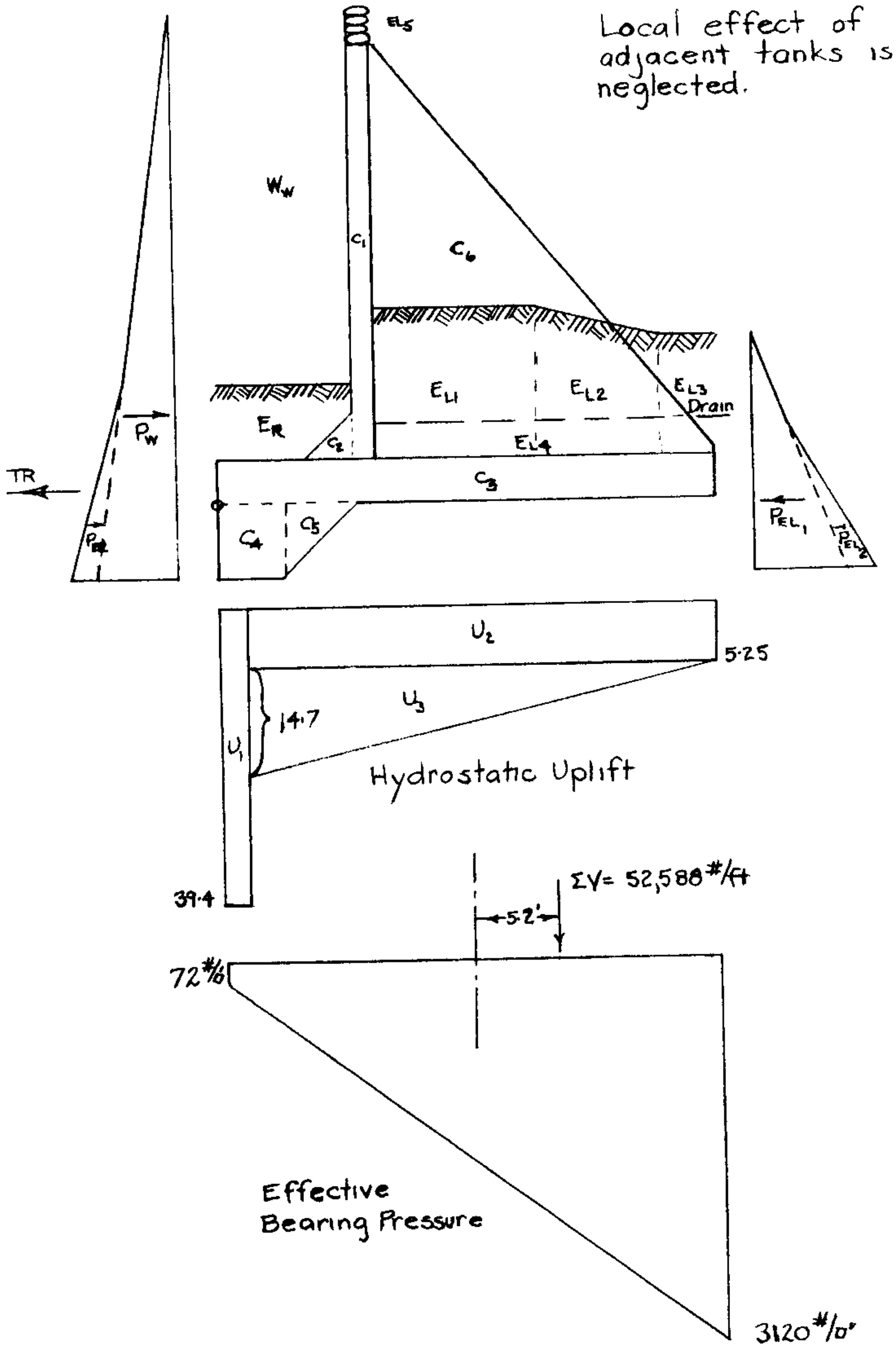
Page

Subject East Hartford - Shell Oil Co - Section "A"

Computation Loading Case I River at flood.

Computed by AWBW Checked by SHB Date 3-7-39

Local effect of adjacent tanks is neglected.



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford – Shell Oil Co – Section "A"

Computation Stability – River at Flood

Computed by [illegible] **Checked by** S.H.B. **Date** 3-10-39

Calculations are per lineal foot of wall.

* indicates an excess, as concrete over earth

Load	Dimensions ft Height × Width	Wt*	Verticals lbs + ↓	Verticals lbs − ↑	Horizontals lbs + →	Horizontals lbs − ←	Lever Arm ft	Moments ft lbs + ↷	Moments ft lbs − ↶
C_1	29.4 × 1.5	150	6,615				9.75	64,500	
C_2	½ × 3.0 × 3.0 *	25	110				8.0	900	
C_3	3.25 × 33.0	150	16,090				16.5	265,490	
C_4	4.75 × 3.5	150	2,490				1.75	4,360	
C_5	½ × 4.75 × 4.75	150	1,690				5.08	8,590	
C_6	½ × 30 × 23 × $\frac{2}{12}$	150	8,625				18.2	156,980	
E_R	7.0 × 9.0	125	7,875				4.5	35,430	
E_{L1}	11.0 × 10.0 × $\frac{10}{12}$	100	9,165				15.5	142,030	
E_{L2}	½(11+8) × 7.5 × $\frac{10}{12}$	100	5,935				24.45	145,110	
E_{L3}	8.0 × 5.0 × $\frac{10}{12}$	100	3,333				30.5	101,660	
E_{L4}	2.0 × 22.5 × $\frac{10}{12}$	125	4,685				21.75	101,900	
E_{L5}	3.0 × 2.0	125	750				10.0	7,500	
W_W	24.4 × 9.0	62.5	13,725				4.5	61,760	
U_1	39.4 × 1.5	62.5		3,695			0.75		2,770
U_2	5.25 × 31.5	62.5		10,335			17.25		178,280
U_3	½ × 14.70 × 31.5	62.5		14,470			12.0		173,640
TR	see pp 3					895	1.25		1,120
P_W	½ × 39.4 × 39.4	62.5			48,510		8.38	406,510	
P_{ER}	½ × 15.0 × 15.0 *	17.5			1,970		0.25	490	
P_{EL_1}	½ × 18.0 × 18.0	35				5670	1.25		7,090
P_{EL_2}	½ × 10.0 × 10.0 *	45				2250	−1.42	3,200	
Σ	Arith		81,088	28,500	50,480	8,815		1,506,410	362,900
	Algeb		52,588		41,665			1,143,510	

$e = \frac{\Sigma M}{\Sigma V} = \frac{1,143,510}{52,588} = 21.7$; Third point $= \frac{2}{3} \times 33 = 22.0$ $e_{℄} = 21.7 - 16.5 = 5.2$

$p = \frac{\Sigma V}{L}\left(1 \pm 6\frac{e}{L}\right) = \frac{52,588}{33}\left(1 \pm \frac{6 \times 5.2}{33}\right) = 1594$ #/□' $(1 \pm .955)$ 3120 #/□' and 72 #/□'

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

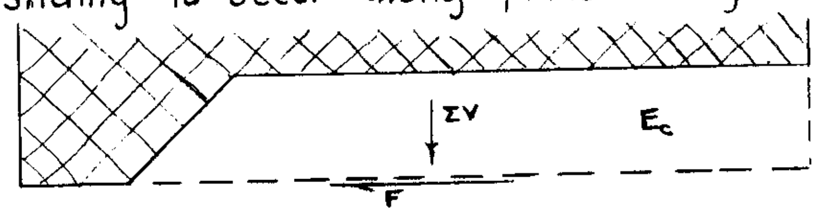
Page

Subject East Hartford-Shell Oil Co - Section "A"

Computation Sliding

Computed by [illegible] **Checked by** S.H.B. **Date** 3-10-39

Assume sliding to occur along plane through lug bottom



Weight of confined earth = $4.75 \times 27.13 \times 125$ =		16,110
Weight of all loads above earth (pp 5)	=	+ 52,588
Total loads normal to shear plane		68,698 #/ft
Total horizontal forces, less resistances		41,665 #/ft
Friction, @ $0.45 \times 68{,}698$		− 30,914
Force to be carried by excess passive		10,751 #/ft

Maximum passive resistance = $\frac{wh^2}{2} \times \frac{(1+\sin\phi)}{(1-\sin\phi)}$

Assume $w = 100$ (on safe side); $\phi = 30°$, $\sin\phi = 0.5$

$$\frac{wh^2}{2} \times \frac{(1+\sin\phi)}{(1-\sin\phi)} = \frac{100 \times 18^2}{2} \times \frac{1.5}{0.5} = 48{,}600 \text{ \#/ft}$$

Excess passive resistance = Maxim. pass. − active used

$$= 48{,}600 - \underbrace{7{,}920}_{P_{E_1}+P_{E_2}} = 40{,}680 \text{ \#/ft}$$

and exceeds that required (10,751 #/ft)

WAR DEPARTMENT

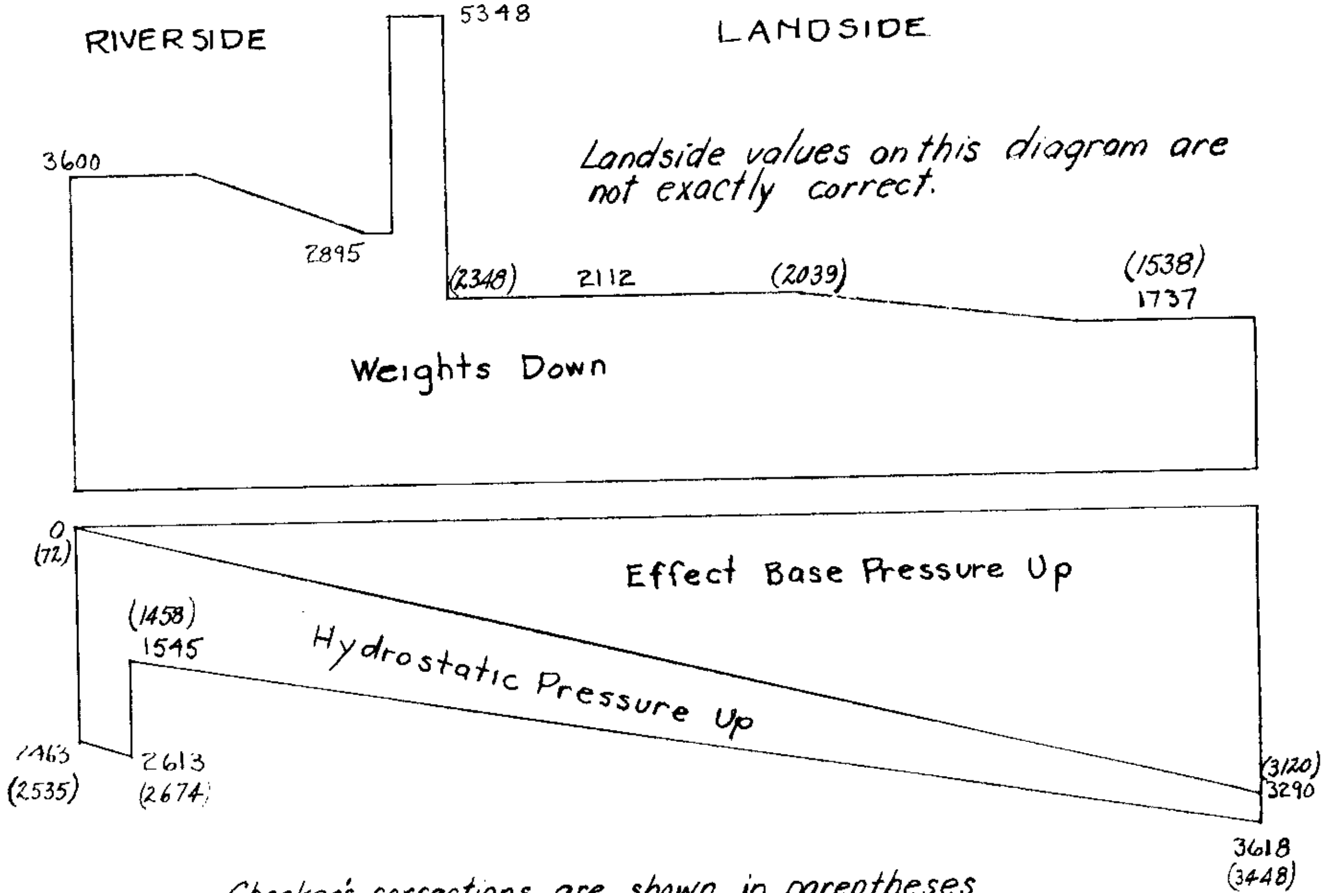
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

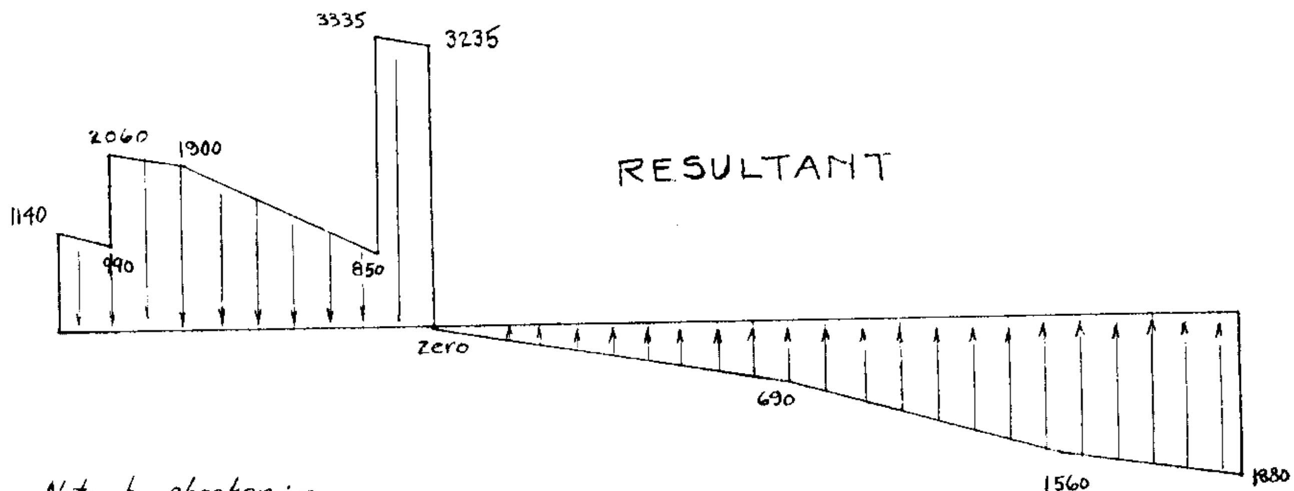
Subject East Hartford - Shell Oil Co - Section "A"

Computation Vertical forces - in #/□'

Computed by [illegible] **Checked by** S.H.B. **Date** 3-10-[illegible]



Checker's corrections are shown in parentheses



Note by checker :-
The base pressure diagram shown on this sheet does not conform exactly to the one shown on sheet 11
Resultant values are approximately correct.

Subject: East Hartford - Shell Oil Co - Section "A"
Computation: Reinforcing Steel - Wall and Base
Computed by: CWKM Checked by: S.H.B. Date: 3-14-39

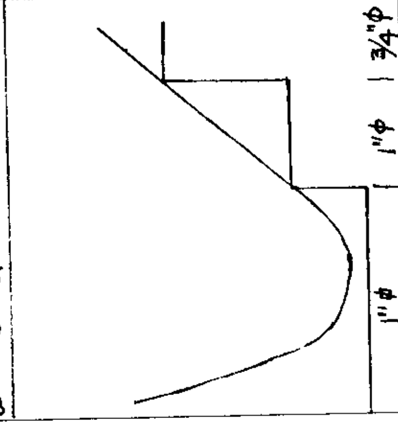
Assume slabs to act as beams fixed at counterforts
Assume maximum moments (a) in wall $wL^2/10$ (b) in base $wL^2/12$

$j = 0.88$; $f_s = 18{,}000$; $f_c = 800$; $v = 60^{\#}/\square''$; $u = 200^{\#}/\square''$

$M = wL^2/10$ & $wL^2/12$; $A_s = \dfrac{M}{f_s j d}$ $V = \dfrac{wL'}{2}$; $v = \dfrac{V}{bjd}$; $u = \dfrac{V}{\Sigma_o jd}$

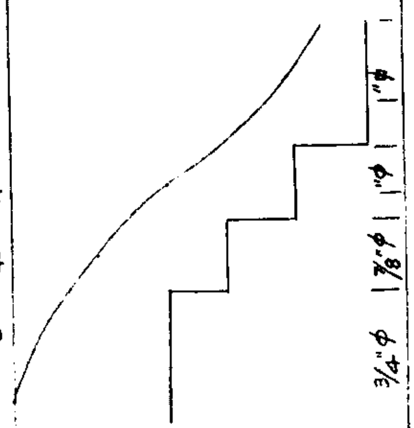
Wall - h represents distance below top of wall

h	pressure #/sq ft River	Land	Diff	M---ft lbs	d "	Steel Area □" A_s	V---lbs	v #/□"	u #/□"
5	437.5	0	437.5	6,300	14.5	0.33	2,190	14	73
10	750.0	0	750	10,800	14.5	0.56	3,750	25	93
15	1062.5	0	1062.5	15,300	14.5	0.80	5,312	35	133
20	1375.0	126	1249	17,990	14.5	0.94	6,245	41	122
25	1733.0	301	1432	20,620	17.5	0.89	7,160	39	116
30	21330	593	1540	22,176	50	0.33	7,700	15	44



Base - x represents distance from outer edge

x	pressures #/□' Up	Down	Diff	M	d	A_s	V	v	u
0	3618	1737	1880	22,560	20.5	0.83	9,400	44	130
5	3297	1737	1560	18,720	20.5	0.69	7,800	36	108
10	3000	2000	1000	12,000	20.5	0.44	5000	23	88
15	2650	2112	540	6,480	20.5	0.24	2700	13	55
20	2300	2112	190	2,280	20.5	0.09	950	5	22
25	2112	2112	zero	zero	20.5	0.00	0	0	0



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page ......

Subject East Hartford - Shell Oil Co - Section "A"

Computation Forces across throat

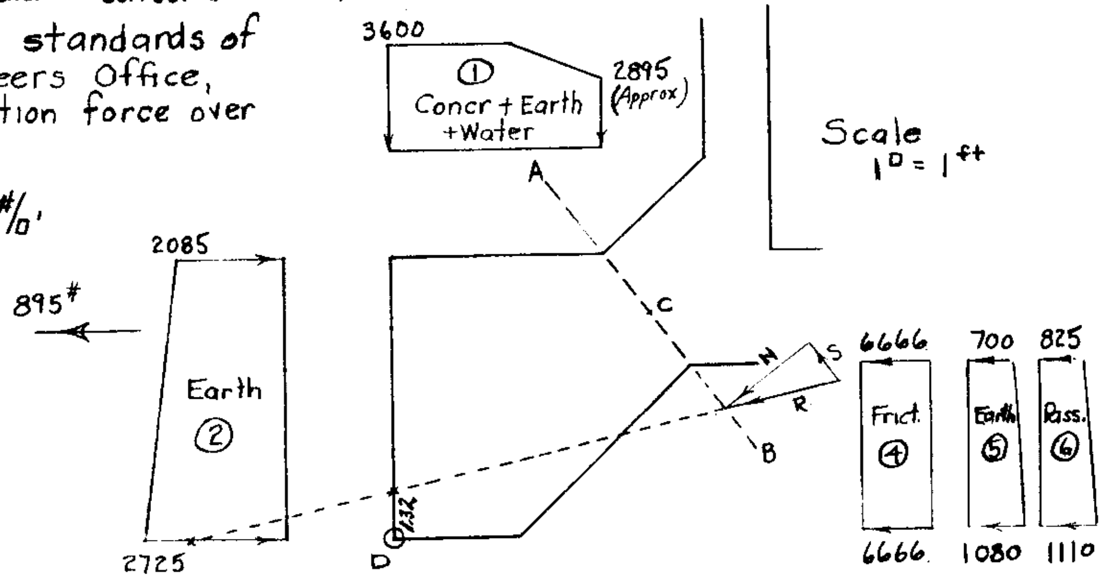
Computed by CWBM **Checked by** L.N.B. **Date** 3-10-39

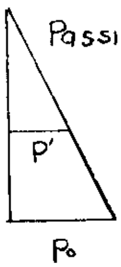
Note by checker :- Corrected value of frictional force has not been carried further.

According to standards of Chief of Engineers Office, spread friction force over entire lug.

(30,914)

$\frac{31,665}{4.75} = 6,666 \#/\square'$





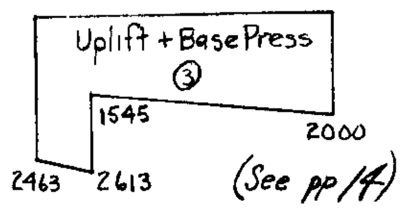
Passive forces (10,751)

$\frac{1}{2} \times P_o \times 18 = 10,000 \#/ft$

$P_o = 1,110 \#/\square'$

$P_1 = 1,110 \times {}^{13.25}/_{18}$

$= 825$



(See pp 14)

Load	Dimensions Height x Width	Verticals lbs + ↓	Verticals − ↑	Horizontals lbs + →	Horizontals − ←	Lever Arm	Moments - D ft lbs +	Moments −
(1)	3600 × 6.0	21,600				3.0	64,800	
	½ × 705 × 2.5		880			5.17		4,550
(2)	2085 × 8.0			16,680		4.0	66,720	
	½ × 640 × 8.0			2,560		2.67	6,825	
	Tie rod pull				(895) 815	6.0		4,890
(3)	(2403) 2400 × 1.5		3,600			0.75		2,700
	½ × 150 × 1.5		110			1.0		110
	1545 × 6.75		10,430			4.88		50,890
	½ × 455 × 6.75		1,535			6.0		9,215
(4)	6666 × 4.75				31,660	2.38		75,360
(5)	700 × 4.75				3,325	2.38		7,910
	½ × 380 × 4.75				900	1.58		1,425
(6)	825 × 4.75				3,920	2.38		9,325
	½ × 285 × 4.75				675	1.58		1,070
Σ	Arith	21,600	16,555	19,240	41,295		138,345	167,445
	Algeb	ΣV = 5,045 ↓			ΣH = 22,055 ←		ΣM_D =	29,100

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Co - Section "A"

Computation Forces across throat

Computed by Curran Checked by A.N.B. Date [illegible]

Intercepts of resultant ---- plot to find line of force

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{-29{,}100}{+\ 5{,}045} = -5.77^{ft} \qquad \bar{y} = \frac{\Sigma M}{\Sigma H} = \frac{-29{,}100}{-22{,}055} = +1.32$$

Shift moment to center of neck.

$$\Delta y = \tfrac{1}{2}(8 + 4.75) = 6.38 \ ; \qquad \Delta x = \tfrac{1}{2}(6 + 8.25) = 7.13$$

$$\Delta M_y = \Delta y \cdot \Sigma H = 6.38 \times 22{,}055 = +140{,}710$$

$$\Delta M_x = \Delta x \cdot \Sigma V = 7.13 \times 5{,}045 = -35{,}970$$

$$M_D = -29{,}100$$

$$M_C = +75{,}640 \ ^{ft\#/ft}$$

Resultant force $= \sqrt{\Sigma V^2 + \Sigma H^2} = \sqrt{5{,}045^2 + 22{,}055^2} = 22{,}625^{\#}$

Check on plotted force line $e_o = \frac{\Sigma M_c}{R} = \frac{75{,}640}{22{,}625} = 3.34'$ (which checks)

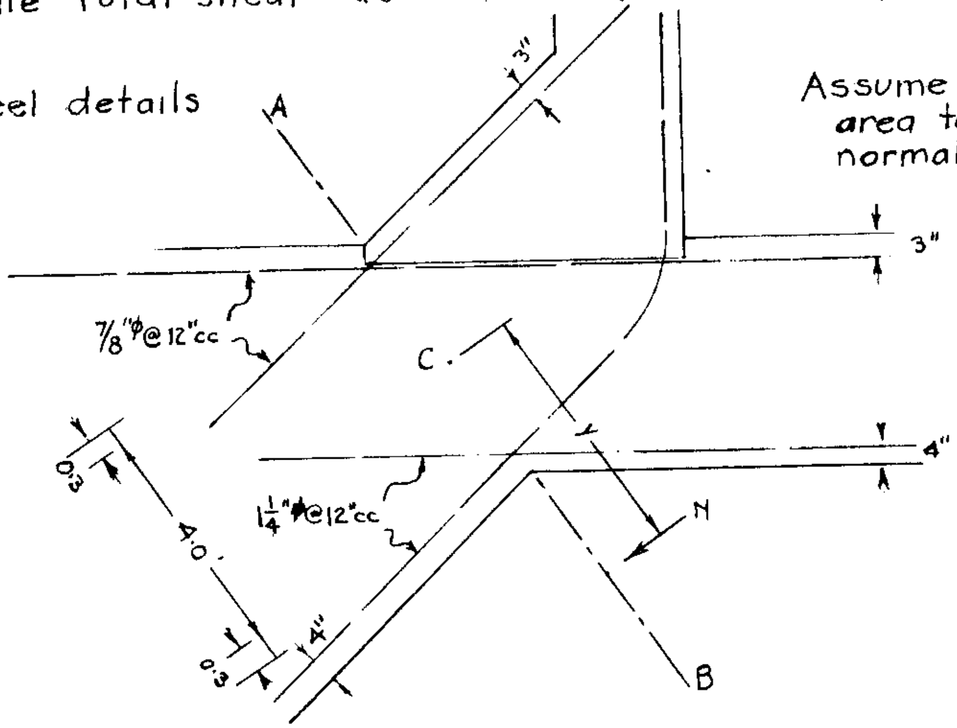
Scale normal thrust as $36/39 \times 22{,}625 = 20{,}885\ ^{\#}/ft$

36 & 39 represent scaled distances on sheet 9

Eccentricity of normal thrust $= \frac{\Sigma M}{N} = \frac{75{,}640}{20{,}885} = 3.62^{ft}$ = *43.44 in*

Scale total shear as $16/39 \times 22{,}625 = 9{,}280\ ^{\#}/ft$

Steel details



Assume 80% of steel area to be effective normal to section AB

Subject East Hartford - Shell Oil Co - Section "A"
Computation Unit Stresses across throat (compression steel used)
Computed by EM **Checked by** [signature] **Date** 3-11-39

The method of analysis used here is outlined on page 14, paragraphs 5 and those following, of Bulletin # 215 University of Illinois, by Hardy Cross

The linear unit used is the inch

Member	Dimens.	A	n	A'	$\bar{y}$	$A'\bar{y}$	$I = A'\bar{y}^2 + I_0$	P	y	M
Steel	0.8×2×7/8φ	0.96	12	11.5	20.4	234.6	4,786	+20,885	-43.44	-907,680
	0.8×2×1¼φ	2.50	12	30.0	-20.4	-612.0	12,485			
				41.5		-377.4				
Concrete	Try 9"	108	1	108	19.5	2106	I_0 = 729 41,067			
C+S				149.5		1729				
Subtract Corr'n					11.56		19,988			241,430
A, I, P, M				149.5			39,079	+20,885		-1,149,110
$P/A = \frac{+20,885}{149.5} =$	+139.7	$y_0 =$		$\frac{-P/A}{M/I} = \frac{-139.7}{-29.40} =$		+4.75";	Compr Concr = 24 - 11.56 - 4.75 = 7.69"			
$M/I = \frac{-1,149,110}{39,079} =$	-29.40									
Concrete	Try 7.6"	91.2	1	91.2	20.2	1842.2	439 37,212			
C+S				132.7		1464.8				
Corr'n					11.04		16,171			230,570
A, I, P, M				132.7			38,751	+20,885		-1,138,250
$P/A = \frac{+20,885}{132.7} =$	+157.4	$y_0 =$		$\frac{-P/A}{M/I} = \frac{-157.4}{-29.37} =$		+5.36";	Compr Concr = 24 - 11.04 - 5.36 = 7.60"			
$M/I = \frac{-1,138,250}{38,751} =$	-29.37									

$$\text{Unit stresses} = n\left[\frac{P}{A} + \frac{M}{I}y\right]$$

Concrete $= 1\left[+157.4 - 29.37(24 - 11.04)\right] = -\ 223$ #/□" compr

Steel 7/8"φ $= 12\left[+157.4 - 29.37(20.4 - 11.04)\right] = -\ 1,410$ #/□" compr

Steel 1¼"φ $= 12\left[+157.4 - 29.37(-20.4 - 11.04)\right] = +12,970$ #/□" tension

Check -

Member	Area	Stress	Force	Lever	Moment
Concrete	½ × 91.2	-223	- 10,169	+ 21.47	-218,328
Steel 7/8φ	0.96	-1410	- 1,354	+ 20.4	- 27,622
1¼ φ	2.50	+12,970	+ 32,425	- 20.4	-661,470
Total (Internal)			+ 20,902		-907,420
External Loads			+ 20,885		-907,680

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. Page .

Subject East Hartford - Shell Oil Co - Section A

Computation Unit stresses across throat (compression steel neglected)

Computed by Curtis **Checked by** A.H.B. **Date** 3-11-29

Member	Dimens	A	n	A'	$\bar{y}$	$A'\bar{y}$	$I = A'\bar{y}^2 + I_0$	P	y	M
Steel	0.8x2x1¼"	2.50	12	300	-20.4	-612.0	12,485	+20,885	-43.44	-907,680
							I_0=729			
Concrete	Try 9"	108	1	108	19.5	2106	41,067			
C+S				138		1494				
Subtract Corr'n					10.83		16,180			226,185
A,I,P,M				138			38,101	+20,885		-1,133,865
$P/A = \frac{+20,885}{138}$ =		+151.3								
$M/I = \frac{-1,133,865}{+38,101}$ =		-29.76	Y_0 =	$\frac{-P/A}{M/I} = \frac{-151.3}{-29.76}$ =		+5.08;	Compr Concr = 24 − 10.83 − 5.08 = 8.09			
Concrete	Try 8.07"	96.84	1	96.84	19.97	1933.9	526 38,620			
C+S				126.84		1321.9				
Corr'n					10.42		13,774			217,622
A,I,P,M				126.84			37,857	+20,885		-1,125,302
$P/A = \frac{+20,885}{126.84}$ =		+164.65								
$M/I = \frac{-1,125,302}{37,857}$ =		-29.73	Y_0 =	$\frac{-P/A}{M/I}$ =	$\frac{-164.65}{-29.73}$	= +5.54	Compr Concr = 24 − 10.42 − 5.54 = 8.04			

$$\text{Unit Stresses} = n\left[\frac{P}{A} + \frac{M}{I}y\right]$$

$$\text{Concrete} = 1\left[+164.65 - 29.73(24 - 10.42)\right] = -239\ \#/\square''$$

$$\text{Steel} = 12\left[+164.65 - 29.73(-20.4 - 10.42)\right] = +12,970\ \#/\square''$$

Check

Member	Area	Stress	Force	Lever	Moment	
Concrete	½x96.84	-239	- 11,572	21.31	-246,599	
Steel	2.50	+12,970	+ 32,425	-20.4	-661,470	
Total			+ 20,853		-908,069	
External Loads			+ 20,885		-907,680	

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. Page

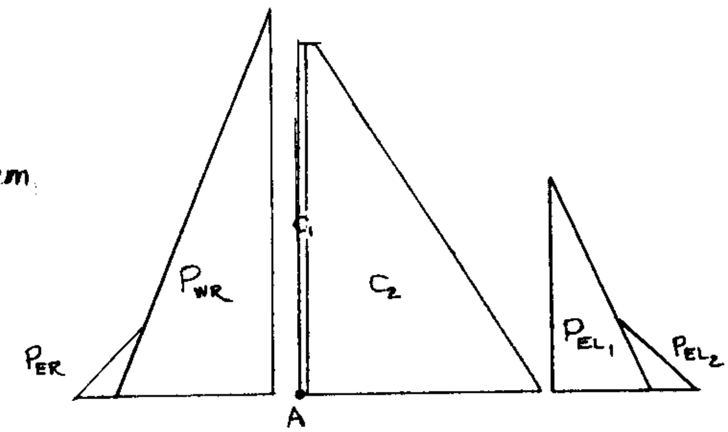
Subject East Hartford - Shell Oil Co - Section A

Computation Stresses along lowest construction joint

Computed by [illegible] Checked by [illegible] Date 3-16-37

Computations are for 12'-0" or one ctft space

Moments are about lower riverside corner of wall stem the haunch being neglected



Item	Dimensions Ht× Wdth× Lgth	ft-# Wt	Verticals lbs + ↓	Verticals lbs − ↑	Horizontals lbs + →	Horizontals lbs − ←	Lever ft Arm	Moments-A- ft lbs + ↷	Moments-A- ft lbs − ↶
C_1	29.4×1.5×12.0	150	79,380				0.75	59,535	
C_2	½×30×23×2.0	150	103,500				9.16	948,060	
P_{WR}	½×31.4×31.4×12	62.5			369,735		10.47	3,871,125	
P_{ER}	½×7.0×7.0×12*	17.5			5,145		2.33	11,990	
P_{EL_1}	½×13.0×13.0×12	35				35,490	4.33		153,670
P_{EL_2}	½×2.0×2.0×12*	45				1 080	0.67		725
Σ Arith			182,880		374,880	36,570		4,890,710	154,395
Algeb			182,880		338,310			4,736,315	

Horizontal shear $= \frac{\Sigma H}{A} = \frac{338,310^{\#}}{2' \times 22.5' \times 144^{\square''}} = 52\ \#/\square''$ (max all $= 90\#/\square''$)

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page ....

Subject East Hartford – Shell Oil Co – Section A
Computation Stresses along lowest construction joint
Computed by CWBM **Checked by** S.N.B. **Date** 3-16-37

Initial origin of coordinates identical with that on preceding page
Coordinates are scaled from detail sheet.

Item	Dimens	A	n	A'	ȳ	A'ȳ	A'ȳ²+I₀	P	y	M
Steel.	12×1¼"φ			18.72	15	280.8	4212	-182,880		+56,836,000
	2×5/8"φ			0.62	30	18.6	558			
	„8			"	42	26.0	1092			
	"			"	54	33.5	1809			
	"			"	66	40.9	2699			
	"			"	78	48.4	3775			
	"			"	90	55.8	5022			
	"			"	102	63.2	6446			
	"			"	114	70.7	8060			
	"			"	126	78.1	9840			
	"			"	138	85.6	11,813			
	"			"	150	93.0	13,950			
	"			"	162	100.4	16,265			
	"			"	174	107.9	18,775			
	"			"	186	115.3	21,445			
	"			"	198	122.8	24,315			
	"			"	210	130.2	27,340			
	"			"	222	137.6	30,545			
	"			"	234	145.1	33,950			
	"			"	246	152.5	37,515			
	"			"	258	160.0	41,280			
	"			"	270	167.4	45,198			
		for	n=1	31.74		2,234	365,904			
	Total	for	n=12	380.9		26,808	4,390,850			
Concrete	Try 10'	10×12×24	1	2880	228	656,640	149,714,000			
C+S				3261		683,448				
Corr'n					209.6		143,250,000			-38,331,000
A, I, P, M				3261			10,855,000	-182,880		+18,505,000

$P/A = \dfrac{-182,880}{3261} = -56.1$ $\quad y_0 = \dfrac{-P/A}{M/I} = \dfrac{+56.1}{-1.705} = -32.9$ $\quad$ Compr Concr = 288 − 209.6 + 32.9 = +111.3

$M/I = \dfrac{-18,505,000}{10,855,000} = -1.705$

Item	Dimens	A	n	A'	ȳ	A'ȳ	A'ȳ²+I₀	P	y	M
Concrete	Try 110"	110×24	1	2640	233.0	615,120	143,322,000			
C+S				3021		641,928				
Corr'n					212.5		136,410,000			-38,862,000
A, I, P, M				3021			11,303,000	-182,880		-17,974,000

$P/A = \dfrac{-182,880}{3021} = -60.5$ $\quad y_0 = \dfrac{-P/A}{M/I} = \dfrac{+60.5}{-1.59} = -38.1$ $\quad$ Compr Concr = 288 − 212.5 + 38.1 = + 113.6

$M/I = \dfrac{-17,974,000}{11,303,000} = -1.590$

Subject: East Hartford - Shell Oil Company - Section A
Computation: Stresses along lowest construction joint.
Computed by: [illegible] Checked by: Date: 3-16-39

Item	Dimens	A	n	A'	$\bar{y}$	$A'\bar{y}$	$A'\bar{y}^2+I$	P	y	M
Concrete	Try 114"	114×24	1	2736	231	632,016	145,996,000			
C+S				3117		658,824				
Corr'n					211.4		139,275,000			-38,660,000
A, I, P, M				3117			11,112,000	-182,880		-18,176,000
P/A =	$\frac{-182,880}{3117}$	= -58.7	y_0 =	$\frac{-P/A}{M/I}$ =	$\frac{+58.7}{-1.636}$	= -35.9	Compr Concr	= 288 - 211.4	+ 35.9 =	112.5
M/I =	$\frac{-18,176,000}{11,112,000}$	= -1.636								

Assume average of values for 110" and 114". $\frac{P}{A} = -59.6$ $\frac{M}{I} = -1.613$

Unit stresses $= n\left(P/A + \frac{M}{I}y\right)$

Concrete $= 1\left(-59.6 - 1.613\,(288 - 212.0)\right) = -182$ #/□" compr

Steel (compr) $= 12\left(-59.6 - 1.613\,(270 - 212.0)\right) = -1838$ #/□" compr

(tens.) $= 12\left(-59.6 - 1.613\,(15 - 212.0)\right) = +3098$ #/□" tens.

Check - (Approximate - using sealed values)

Item	Dimensions	Force	Lever	Moment
Concrete	½×112×24×(-182)	-244,600	250.7	- 61,321,000
Steel (Compr)	½×8×0.62×(-1838)	- 4,560	240	- 1,094,000
(Tens.)	½×12×0.62×(+2800)	+ 10,420	78.7	+ 820,000
(Tens.)	18.72×(+3098)	+ 58,000	15	+ 870,000
Total		- 180,740		- 60,700,000
External		- 182,880		- 56,800,000

Vertical shear along junction of wall slab and counterfort

= load carried by 1¼"φ bars = 58,000#

Unit shear $= \frac{V}{A} = \frac{58,000}{2 \times 29.4 \times 144} = 7$ #/□"

Construction joint

Normal force = 244,600 + 4,560 = 249,200#
Friction @ 0.45×249,200 = 112,100#
Horizontal shear = 338,300
Difference = 226,200# to be carried by keys
Each key (4" thick) can carry 4"×24"×500#/□" = 48,000#
No. of keys required = 226,200 ÷ 48,000 = 5

WAR DEPARTMENT

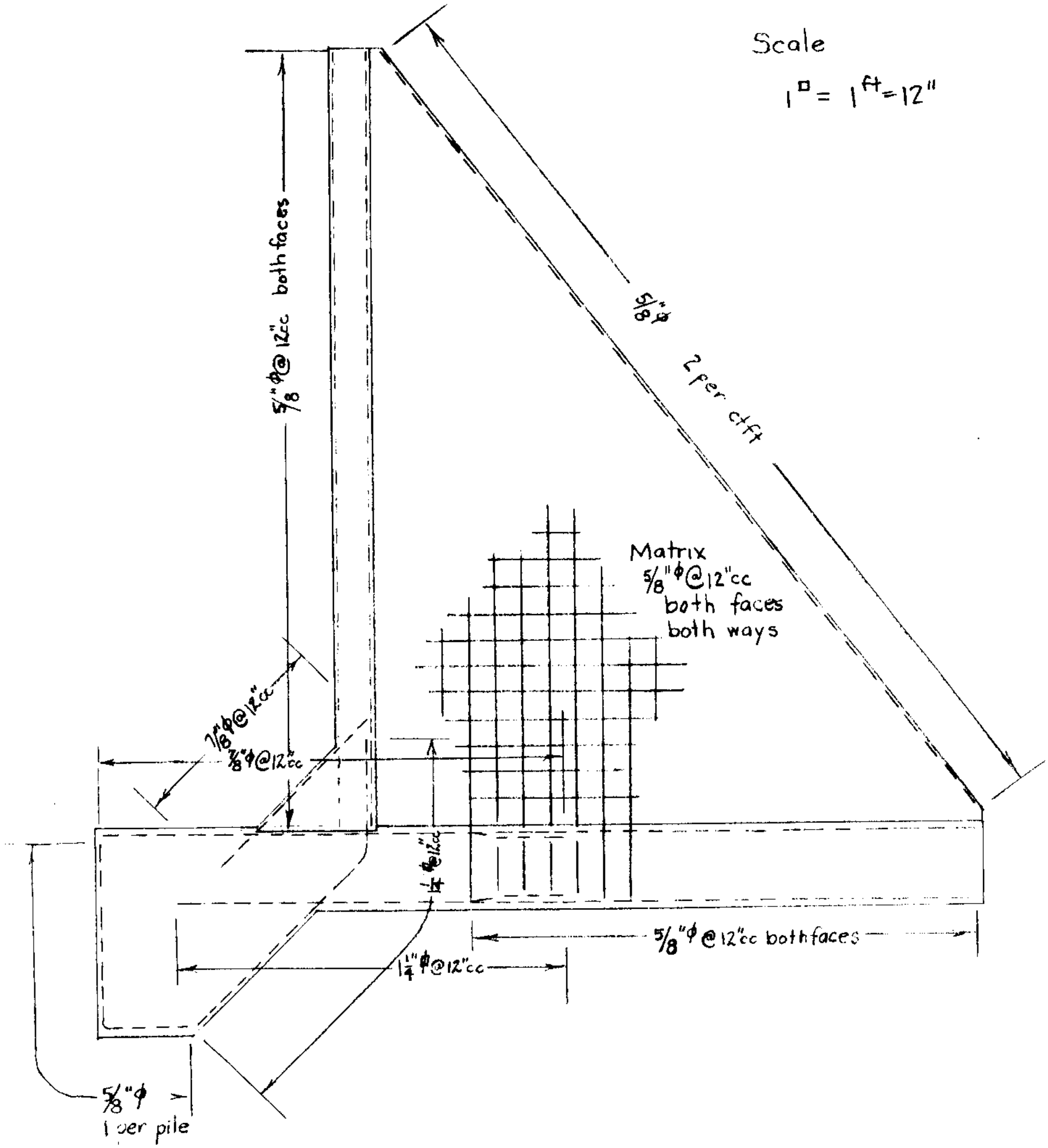
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page ....

Subject: East Hartford - Shell Oil Co - Section "A"

Computation: Transverse Steel

Computed by: [illegible] Checked by: S.H.B. Date: 1-11-39



WAR DEPARTMENT

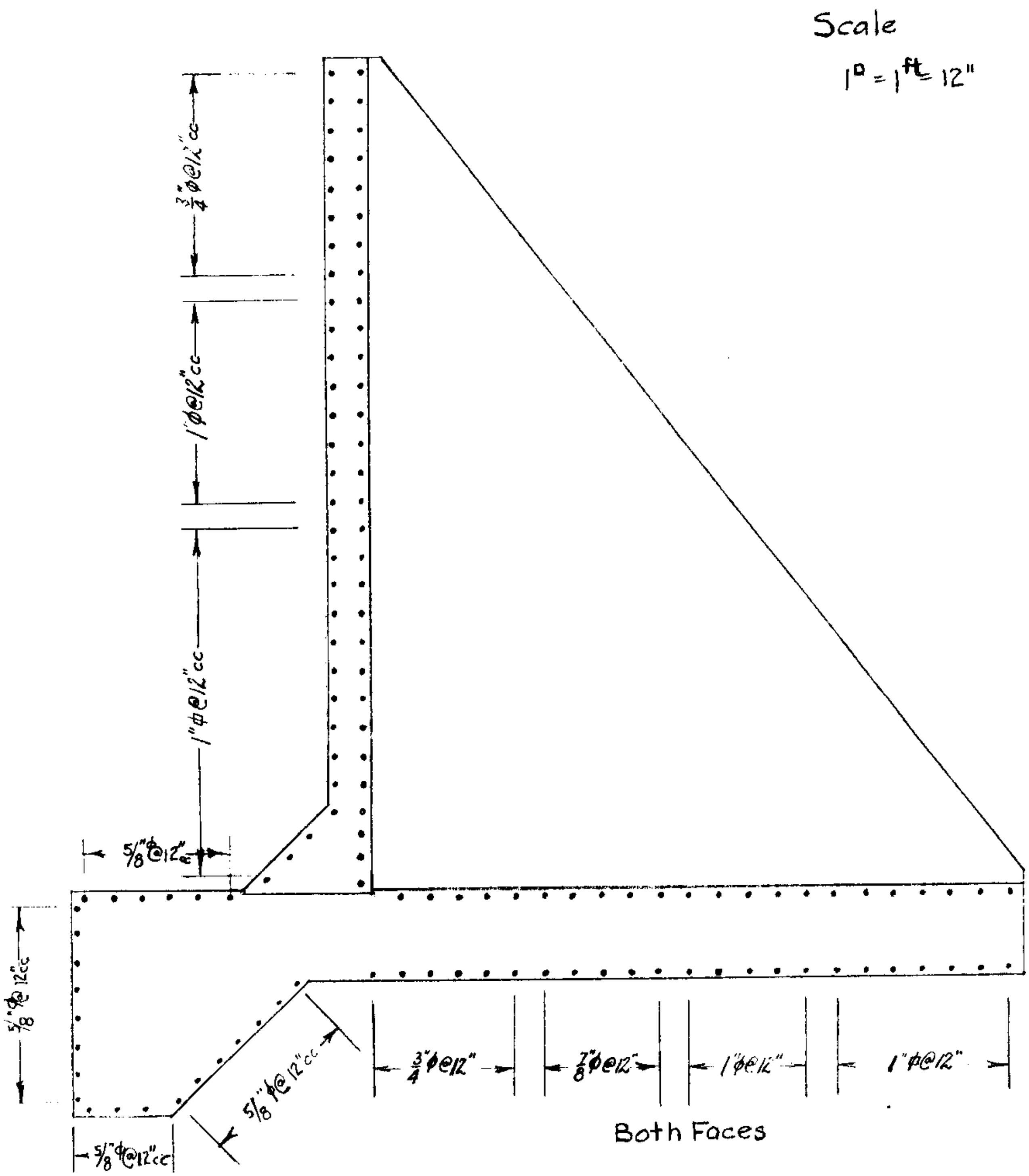
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Co - Section "A"

Computation Longitudinal Steel

Computed by [illegible] Checked by [illegible] Date 1-11-37



WAR DEPARTMENT

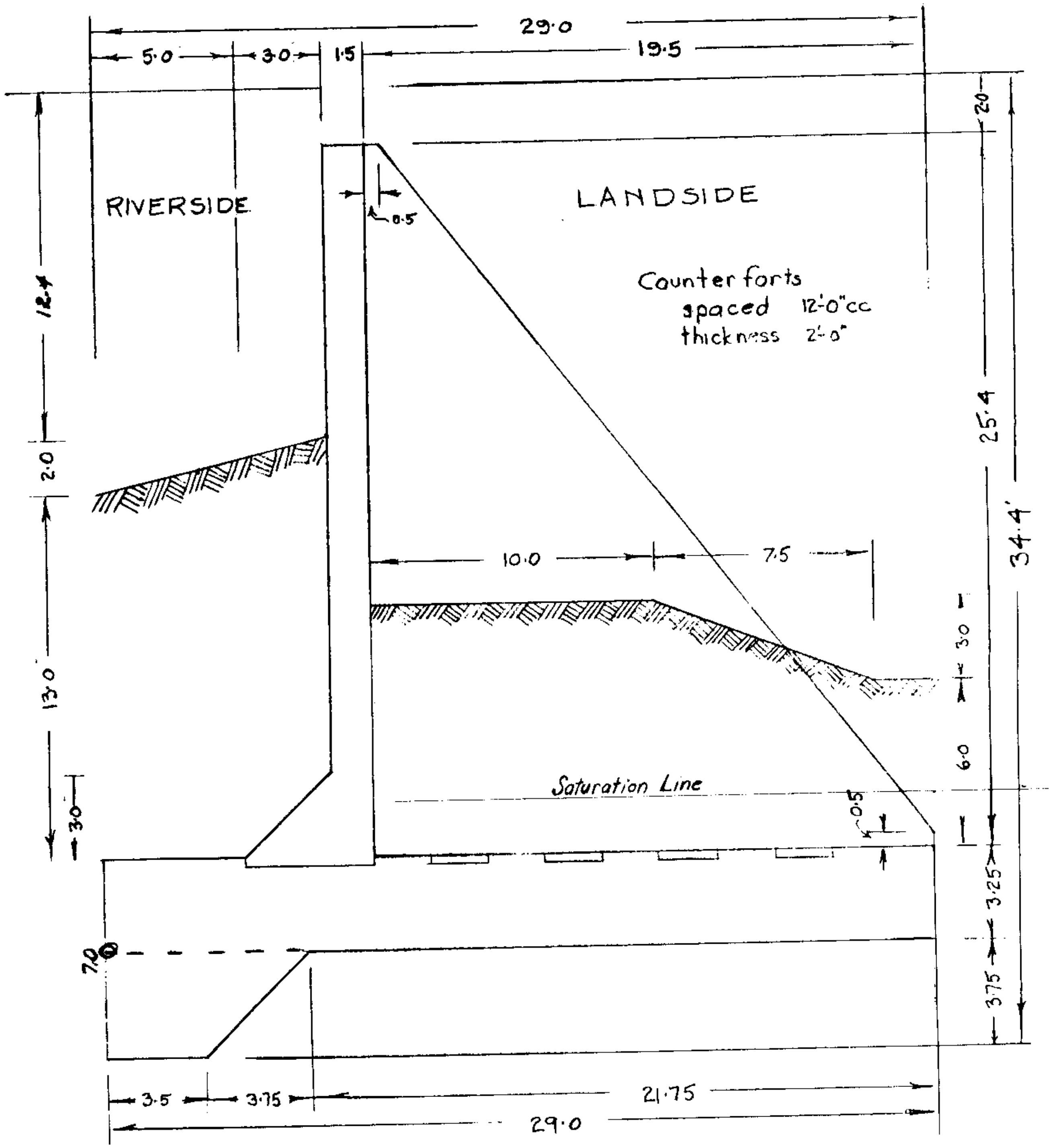
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Company - Section C

Computation

Computed by [illegible] Checked by S.H.B. Date 3-1-37



WAR DEPARTMENT

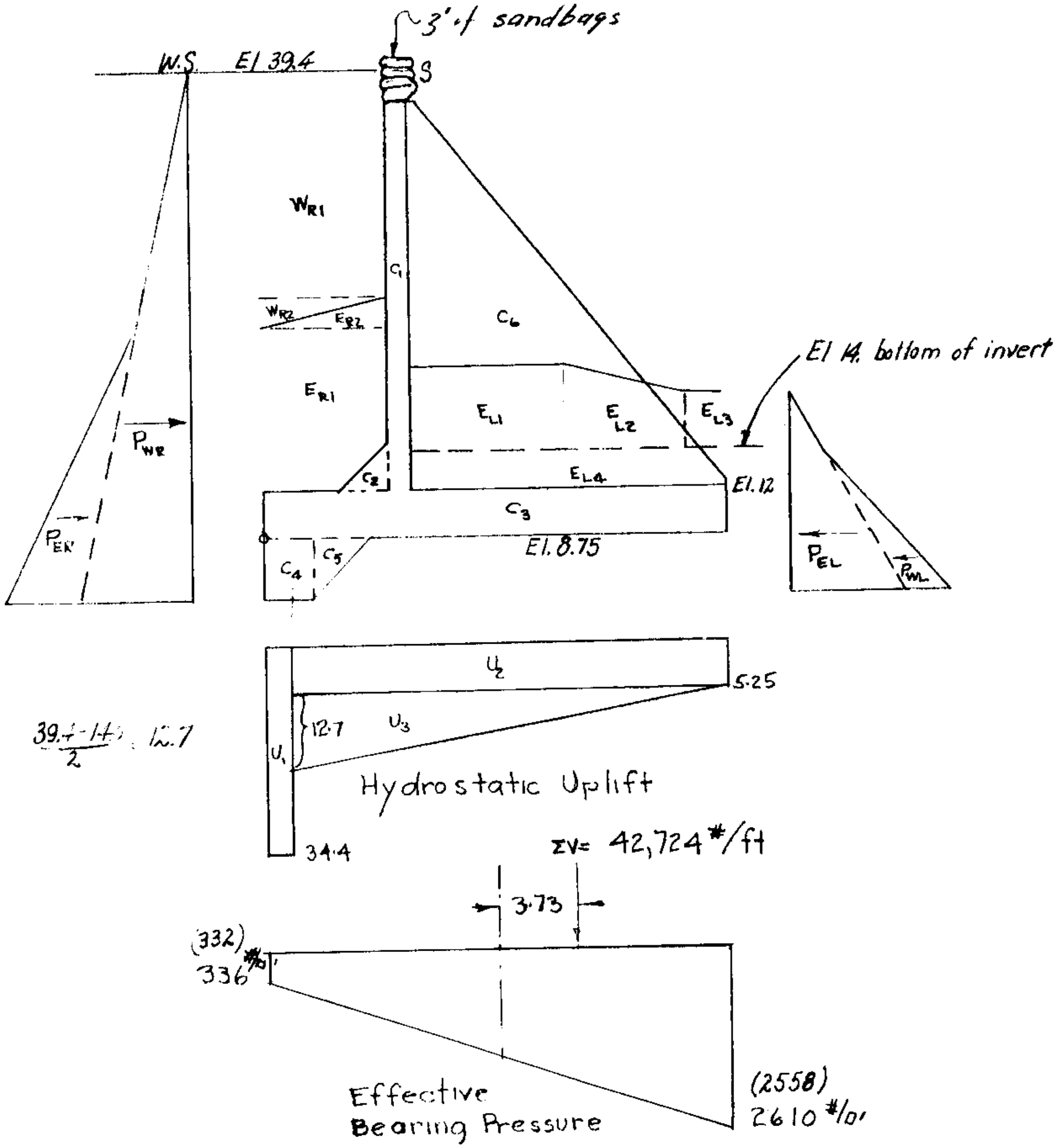
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Co - Section C

Computation Loadings for stability

Computed by ... Checked by ... Date 3-9-39



Corrections are in parentheses

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Co - Section C

Computation Stability - River at Flood - Tailwater

Computed by CWBM **Checked by** AWB **Date** 3-17-39

Calculations are per lineal foot

Item	Dimensions Ht × Width ft	Wt#	Verticals lbs + ↓	− ↑	Horizontals + →	− ←	Lever Arm	Moments - ft lbs + ↷	− ↶
C_1	25.4×1.5	150	5,715				8.75	50,000	
C_2	$\frac{1}{2}$×3.0×3.0 *	25	113				7.0	790	
C_3	3.25×29.0	150	14,138				14.5	205,000	
C_4	3.75×3.5	150	1,969				1.75	3,445	
C_5	$\frac{1}{2}$×3.75×3.75	150	1,055				4.75	5,010	
C_6	$\frac{1}{2}$×26.0×20×$\frac{2}{12}$	150	6,500				16.33	106,145	
E_{R1}	13.0×8.0	125	13,000				4.0	52,000	
E_{R2}	$\frac{1}{2}$×2.0×8.0	125	1,000				5.33	5,330	
E_{L1}	7.0×10.0×$\frac{10}{12}$	100	5,833				14.5	84,580	
E_{L2}	$\frac{1}{2}$(7+4)×7.5×$\frac{10}{12}$	100	3,438				22.91	78,765	
E_{L3}	4.0×2.0×$\frac{10}{12}$	100	800				28.0	22,400	
E_{L4}	2.0×19.5×$\frac{10}{12}$	125	4,060				19.25	78,155	
S	3×2	125	750				9.0	6,750	
W_{R1}	12.4×8.0	62.5	6200				4.0	24,800	
W_{R2}	$\frac{1}{2}$×2.0×8.0	62.5	500				2.67	1,335	
P_{WR}	$\frac{1}{2}$×34.4×34.4	62.5			36980		7.71	285,115	
P_{ER}	$\frac{1}{2}$·20.0×20.0 *	17.5			3,500		2.92	10,220	
P_{EL}	$\frac{1}{2}$×13.0×13.0	35				2,958	0.58		1,715
P_{WL}	$\frac{1}{2}$×9.0×9.0 *	45				1,823	−0.75	1,365	
U_1	34.4×1.5	62.5		3,225			0.75		2,420
U_2	5.25×27.5	62.5		9,023			15.25		137,600
U_3	$\frac{1}{2}$×12.7×27.5	62.5		10,914			10.67		116,450
Σ	Arith		65,071	23,162	40,480	4,781		1,021,205	258 185
	Algeb		41,909		35,699			763,020	

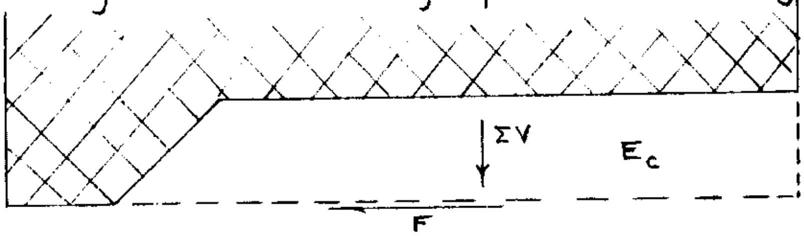
$$e = \frac{\Sigma M}{\Sigma V} = \frac{+763,020}{+41,909} = +18.21 \qquad \text{Third point} = \frac{2}{3} \times 29.0 = 19.33'$$

$$e_{\not{c}} = 18.21 - \frac{29.0}{2} = 3.71$$

$$p = \frac{\Sigma V}{A}\left(1 \pm 6 \frac{e_{\not{c}}}{L}\right) = \frac{41,909}{29}\left(1 \pm 6 \times \frac{3.71}{29.0}\right) = 1445\left(1 \pm 0.77\right) = \begin{cases} 2558\ \#/\square' \\ 332\ \#/\square' \end{cases}$$

Subject East Hartford - Shell Oil Co - Section C
Computation Sliding
Computed by [illegible] Checked by [illegible] Date

Assume sliding to occur along plane thru lug bottom



Weight of confined earth = 3.75 × 23.0 × 125′	=	10,781
Weight of all loads above earth (pp 3)	=	41,909
Total Loads normal to shear plane	=	52,690
Total horizontal forces (pp 3)	=	35,699 →
Friction, @ 0.45 × 52,690	=	23,710 ←
Force to be carried by excess passive		11,989′ #/ft

Maximum passive resistance = $\frac{wh^2}{2}\frac{(1+\sin\phi)}{(1-\sin\phi)}$

Assume $w = 100$ (safe side); $\phi = 30°$; $\sin\phi = 0.5$

$$\frac{wh^2}{2}\frac{(1+\sin\phi)}{(1-\sin\phi)} = \frac{100\times 13^2}{2}\times\frac{(1.5)}{(0.5)} = 25,400^{\#}$$

Excess passive resistance = Maxim pass. — active used
= 25,400# − 4,780# = 20,620′ #/ft

and exceeds that required (11,990 #/ft)′

WAR DEPARTMENT

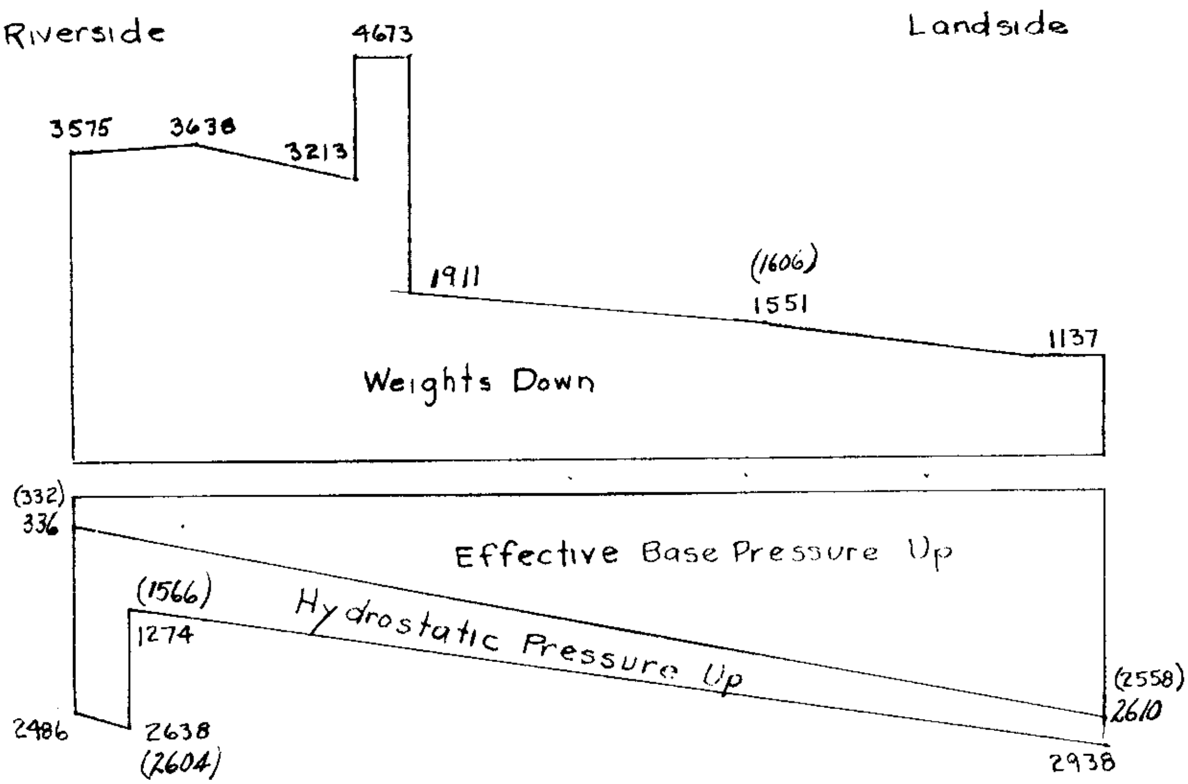
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

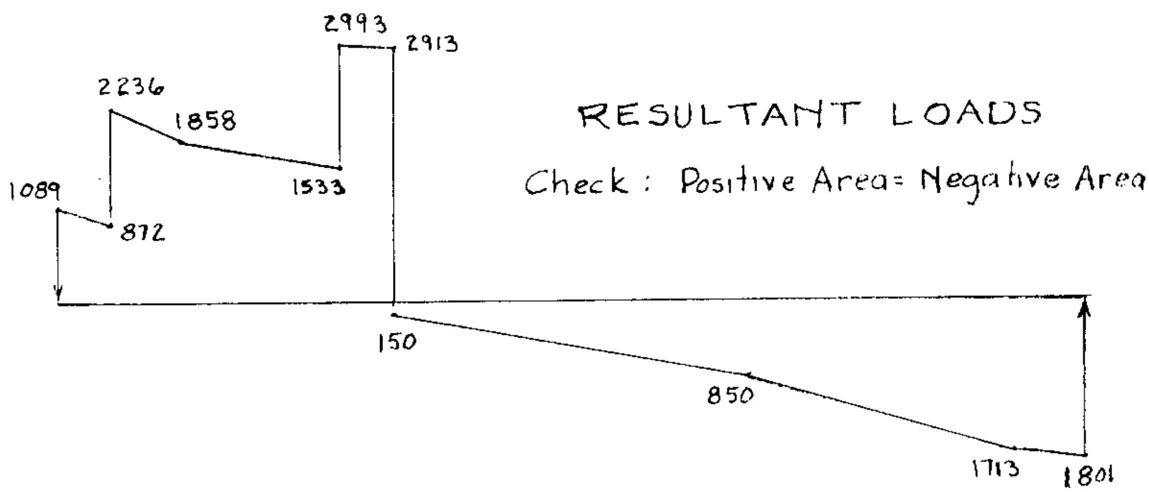
Subject East Hartford- Shell Oil Co. Section C

Computation Effective loads on base slab (in lbs per sq ft)

Computed by CWEM + JSM Checked by SHB Date 3-15



Note by checker :-
Corrections are not carried further.



Note by checker :-
Resultant pressures are approximately correct, except for changes due to corrections on first two diagrams on this sheet.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page ....

Subject: East Hartford - Shell Oil Co - Section C

Computation: Reinforcing Steel -

Computed by: [illegible] Checked by: [illegible] Date: 3-14-39

Assume slabs to act as beams fixed at counterforts

Assume maximum moments (a) in wall $\omega L^2/10$ (b) in base $\omega L^2/12$

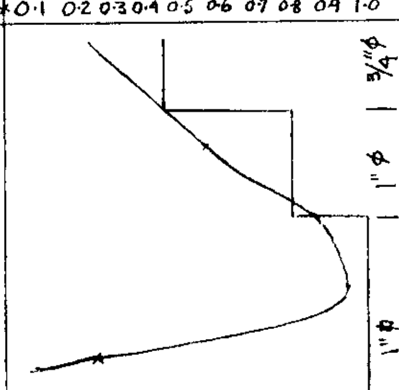
$L = 12$

$j = 0.88$ $f_s = 18{,}000$ $f_c = 800$ $v = 60\#/\square''$ $u = 200\#/\square''$

$A_s = M/f_s j d$ $V = \omega L'/2$ ($L' = 10$) $v = V/bjd$ $u = V/\Sigma_0 jd$

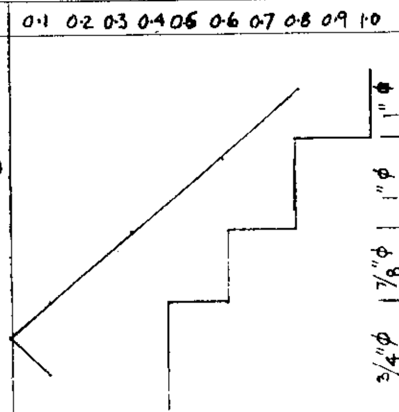
Wall - h represents distance below top of wall

h	pressure #/□' River	pressure #/□' Land	pressure #/□' Diff	M - ft#	d"	Steel Area □" A_s	V - #	v #/□"	u #/□"
5	437.5	0	437.5	6,300	14.5	0.33	2190	15	73
10	750	0	750	10,800	14.5	0.56	3750	25	94
15	1143	0	1143	16,460	14.5	0.86	5715	38	145
20	1543	288	1255	18,070	14.5	0.94	6275	41	123
25	1943	688	1255	18,070	50.0	0.27	6275	12	35



Base - x represents distance from outer edge

x	pressure #/□' Down	pressure #/□' Up	pressure #/□' Diff	M - ft#	d"	Steel Area A_s	V #	v #/□"	u #/□"
0	1137	2938	1801	21,612	20.5	0.80	9000	42	125
5	1300	2620	1320	15,840	20.5	0.59	6600	31	116
10	1580	2320	740	8,880	20.5	0.33	3700	17	75
15	1770	2020	250	3,000	20.5	0.11	1250	6	29
20	1950	1710	240	2,880	20.5	0.11	1200	6	28



$\underbrace{\text{Down, Up}}_{\text{Scaled Values (approximate)}}$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Co - Section C

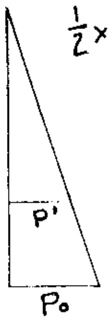
Computation Forces across throat

Computed by [illegible] **Checked by** S.W.B. **Date** 3-15-39

By recommendations of Office of Chief of Engineers, spread friction uniformly over lug

$$\frac{24{,}077}{3.75} = 6{,}420 \ \#/\square'$$

Passive forces



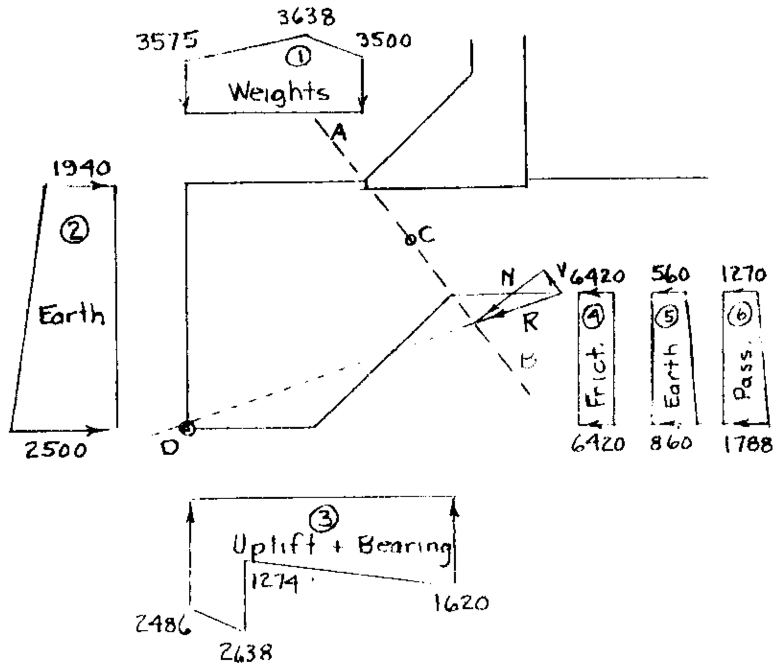
$$\tfrac{1}{2} \times p_o \times 13 = 11{,}622$$

$$p_o = 1{,}788 \ \#/\square'$$

$$p_1 = 1{,}788 \times \frac{9.25}{13}$$

$$= 1{,}270 \ \#/\square'$$

Scale 1" = 1'



Item	Dimensions	Verticals lbs + ↓	Verticals lbs − ↑	Horizontals lbs + →	Horizontals lbs − ←	Lever Arm	Moments D- ft lbs + ↺	Moments D- ft lbs − ↻
(1)	3638 × 5.0	18,190				2.5	45,475	
	½ × 63 × 3.5		110			1.75		193
	½ × 138 × 1.5		104			4.5		468
(2)	1940 × 7.0			13,580		3.5	47,530	
	½ × 560 × 7.0			1,960		2.33	4,567	
(3)	2486 × 1.5		3,729			0.75		2,797
	½ × 152 × 1.5		114			1.0		114
	1274 × 5.75		7,326			4.38		32,088
	½ × 346 × 5.75		995			5.33		5,303
(4)	6420 × 3.75				24,075	1.88		45,261
	560 × 3.75				2100	1.88		3,948
	½ × 300 × 3.75				563	1.25		704
	1270 × 3.75				4763	1.88		[illegible]
	½ × 518 × 3.75				971	1.25		1,214
Σ	Arithm.	18,190	12,378	15,540	32,472		97,572	101,044
	Algebr.	5,812			16,932			3,472

Intercepts of resultant - - - - - plot to find line of force

$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{-3,472}{5,812} = -0.60$ $\bar{y} = \frac{\Sigma M}{\Sigma H} = \frac{-3,472}{-16,932} = +0.21$

An additional point - $x = 10$; $y = +0.21 + \frac{5,812}{16,932} \times 10 = 3.64$

Shift moment to center of neck

$\Delta x = \frac{1}{2}(5 + 7.25) = 6.13$ $\Delta y = \frac{1}{2}(7 + 3.75) = 5.38$

$\Delta M_x = \Delta x \cdot \Sigma V = 6.13 \times 5812 = -35,628$

$\Delta M_y = \Delta y \cdot \Sigma H = 5.38 \times 16,932 = +91,094$

$M_D = -3,472$

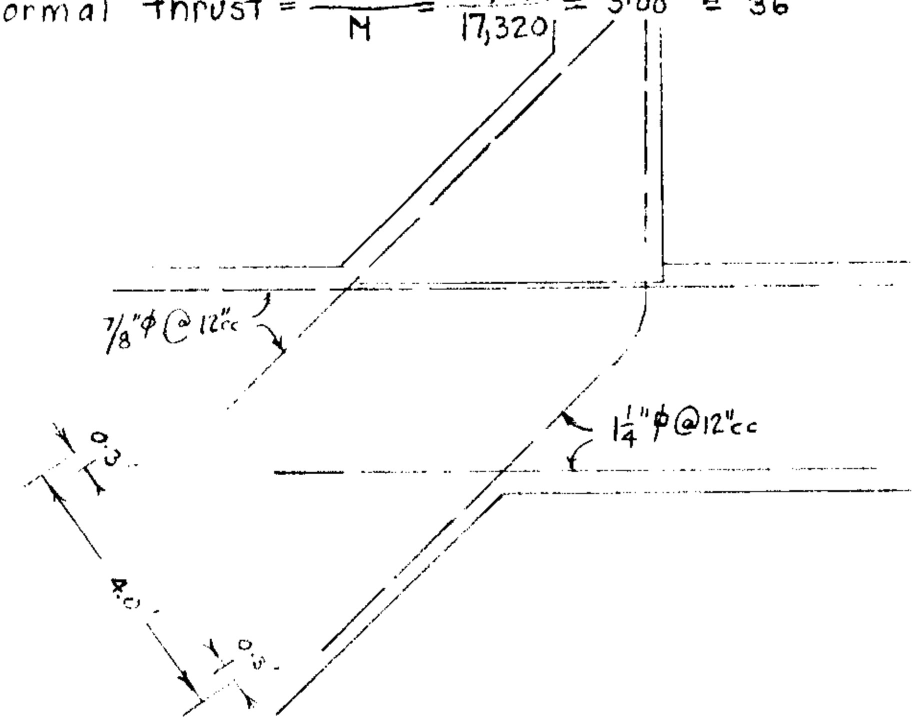
$M_C = +51,994$

Resultant force $= \sqrt{\Sigma V^2 + \Sigma H^2} = \sqrt{5,812^2 + 16,932^2} = 17,900\# = R$

Normal thrust (scaled) $= 17,900 \times \frac{30}{31} = 17,320\# = N$

Shear (total) (scaled) $= 17,900 \times \frac{10}{31} = 5,775\# = V$

Eccentricity of normal thrust $= \frac{\Sigma M}{N} = \frac{51,994}{17,320} = 3.00' = 36''$



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — Page ....

Subject East Hartford - Shell Oil Co - Section C

Computation Stresses across throat (compression steel used)

Computed by SHB **Checked by** CWEM **Date** 3-16-39

The method used here is outlined on pp 14 and those that follow, Bulletin # 215, University of Illinois Bulletin by Hardy Cross.

Linear unit used is the inch

Member	Dimens	A	n	A'	$\bar{y}$	$A'\bar{y}$	$A'\bar{y}^2 + I_o$	P	y	M
Steel	$0.8\times2\times\frac{7}{8}''\phi$	0.96	12	11.5	20.4	234.6	4,785	+17,320	-36.0	-623,930.
	$0.8\times2\times1\frac{1}{4}''\phi$	2.50	12	30.0	-20.4	-612.0	12,485			
				41.5		-377.4	17,270			
Concrete	Try 6"	72	1	72	21	1,512	31,752			
C+S				113.5		1134.6				
Corr'n					10.0		11,346			173,200
A,I,P,M				113.5			37,676	+17,320		-797,130
P/A =	$\frac{+17,320}{113.5}$	= 152.5								
M/I =	$\frac{-797,130}{37,676}$	= -21.1	y_o =	$\frac{-P/A}{M/I}$ =	+7.24		Compr Concr = 24-10.0-7.24			= 6.76"
Concrete	Try 6.7"	80.4	1	80.4	20.65	1660.	34,300			
C+S				121.9		1283.				
Corr'n					10.53		13,500			182,400
A,I,P,M				121.9			38,070	+17,320		-806,330
P/A =	$\frac{17,320}{121.9}$	= 142.2								
M/I =	$\frac{-806,330}{38,070}$	= -21.15	y_o =	$\frac{-P/A}{M/I}$	= 6.73		Compr Concr = 24-10.53-6.73			= 6.74"

Unit Stresses $= n\left[P/A + My/I\right]$

Concrete $= 1\left[142.2 - 21.15(24.0-10.53)\right] = -\ 143$ #/□" compr.

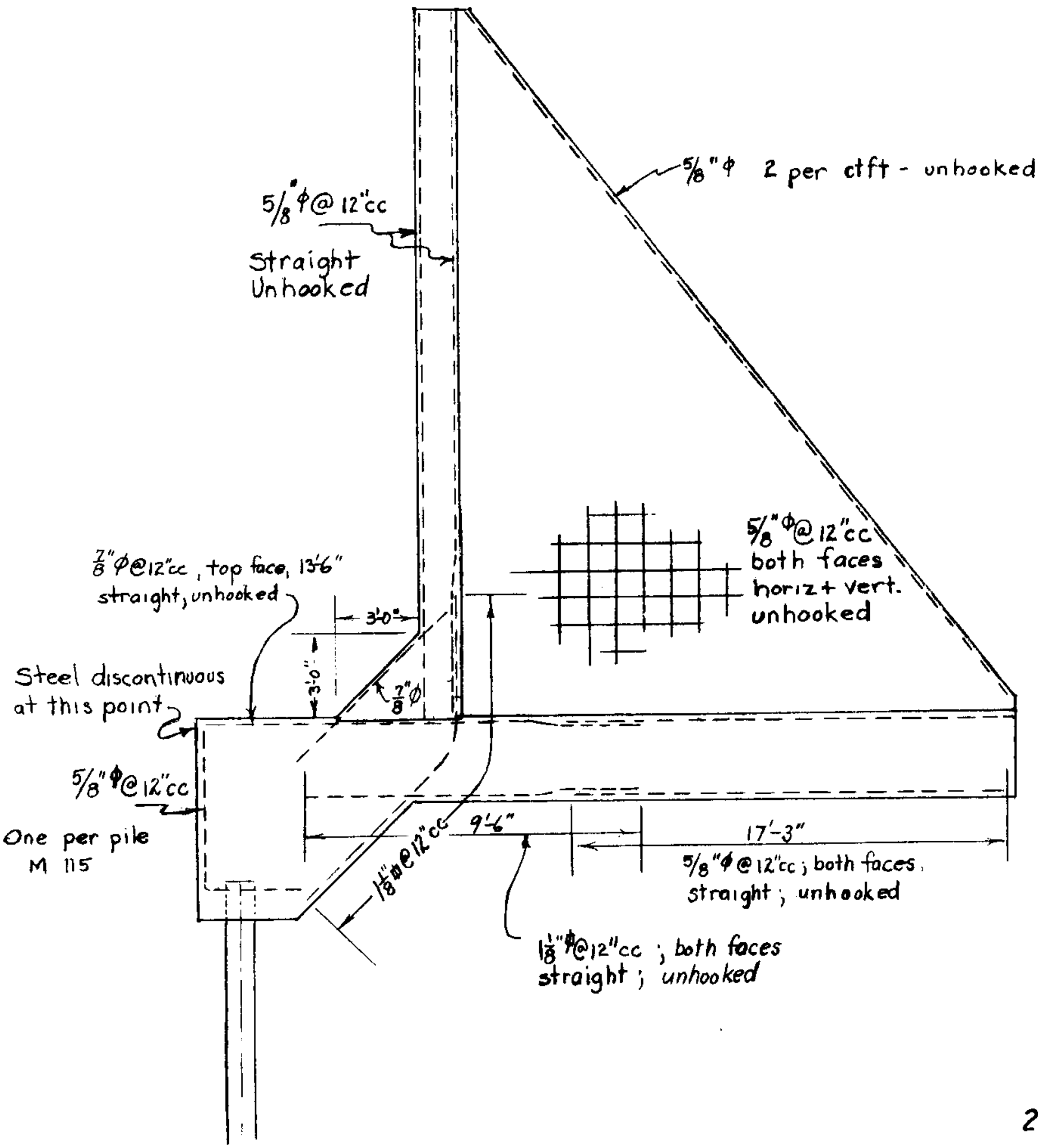
Steel $\frac{7}{8}''\phi = 12\left[142.2 - 21.15(20.4-10.53)\right] = -\ 795$ #/□" compr.

" $1\frac{1}{4}''\phi = 12\left[142.2 - 21.15(-20.4-10.53)\right] = +\ 9{,}560$ #/□" tension

Check

Member	Area	Stress	Force	Arm	Moment
Concrete	$\frac{1}{2}\times 80.4$	-143.	- 5,750.	21.77	- 125,100
Steel $\frac{7}{8}''\phi$	0.96	-795.	- 763.	20.4	- 15,570
Steel $1\frac{1}{4}''\phi$	2.50	+9,560.	+ 23,900.	-20.4	- 487,500
Total			+ 17,387.		- 628,170
External			+ 17,320		- 623,930.

Subject East Hartford - Shell Oil Co - Section "G"
Computation Final Steel Arrangement - Transverse
Computed by WRM Checked by SAK Date 1-8-41



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page

Subject East Hartford - Shell Oil Co - "C"

Computation Longitudinal Steel

Computed by CWBW Checked by SHB Date 1-8-39